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EXPERIMENTAL SECTION OF THE COLUMBIA PIKE SHOWING TYPICAL CRACKS IN A REINFORCED CONCRETE SURFACE

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THE INTERRELATION OF LONGITUDINAL STEEL AND TRANSVERSE CRACKS IN CONCRETE ROADS

Reported by A. T. GOLDBECK, Chief, Division of Tests, U. S. Bureau of Public Roads

IT is evident from a study of standard designs for concrete pavements that there is a great difference of opinion among engineers as to the proper amount and placement of longitudinal steel. A similar lack of agreement appears in the practice with regard to the spacing of joints; and there is considerable evidence of a lack of appreciation of the essential interrelation between the quantity of steel and the spacing of joints.

That there is such a relation has been definitely shown by investigations made by the Bureau of Public Roads, and it would seem that the facts developed are now sufficiently substantiated to serve as the basis for a rational method of design.

As a preface to the discussion of the method which it is intended to propose it is necessary to have a clear understanding of the various forces which act upon the pavement and a knowledge of the physical properties of the steel and concrete which determine their behavior when they are combined.

It will be assumed therefore that a uniform subgrade has been prepared. The concrete has been placed and begins to harden. Immediately the moisture begins to dry out of it and as it dries it shrinks. The surface dries and shrinks more rapidly than the immediately underlying concrete, thus subjecting the upper skin to tensile stresses which will quickly exceed the slight tensile strength of the material and cause numerous fine surface cracks to form unless the process of shrinkage is halted by the timely beginning of curing.

The curing process, by keeping the concrete moist, prevents it from shrinking until it has attained greater strength, and thereby minimizes cracking. The daily wetting not only prevents shrinking but actually causes an expansion which seems to be about 0.0001 inch per inch of length; and as long as the wetting continues a compressive stress is set up which must be counterbalanced before the concrete can be subjected to tension.

The importance of this initial expansion lies in the fact that the concrete, from the moment it is placed, is subjected to alternate compressive and tensile stresses caused by increases and decreases in temperature. The compressive stresses are negligible in their effect, but the tensile stresses would often be sufficient to crack the pavement during the early stages were it not for the initial compression set up by the curing process.

CONTRACTION CRACKS CAUSED BY SUBGRADE FRICTION

All contraction cracks, whether they take the form of surface checking or larger cracks extending through the whole depth of the pavement, result from the effort of the contracting concrete to drag itself over a less rapidly contracting portion of the pavement or over the subgrade. In the case of surface checking it is the lower portion of the concrete itself which, opposing the contraction of the surface skin, sets up the tensile forces that crack the surface. The larger cracks result from frictional resistance to the movement of the whole mass of the pavement over the subgrade.

It will be helpful at this point to analyze in detail the action leading up to the formation of one of the larger cracks. We start with a freshly laid concrete pavement. It dries and tends to contract. If it rested upon a perfectly frictionless base the contraction would take place without causing cracking. The pavement, free to move without any resistance, would draw itself together and set up into a solid, uncracked slab. But this perfect condition does not obtain. The subgrade is not a frictionless plane. It offers more or less frictional resistance to the contractive movement of the pavement.

Now, for each condition of dryness or temperature the pavement has a certain normal length which it would assume if unprevented by external forces. If, under any given condition, it is prevented from assuming the normal length pertaining to that condition it is strained, though actually its length may not change. Thus a drying concrete pavement may be increasingly stressed and strained, although its actual length may remain unchanged. It is the normal length that is changing, and as this becomes increasingly less than the actual length the stress mounts up, and a point is finally reached where the tensile strength of the concrete is exceeded and a crack forms. While this process is going on the concrete may not move at all. Its actual length may remain unchanged. If it does move and thereby shorten its actual length, the stress is relieved.

HOW THE CRACKS FORM IN PLAIN CONCRETE PAVEMENTS

With these facts in mind let us now examine the behavior of our drying concrete pavement. It is built without joints. Originally it extends as an unbroken slab of moist concrete between the free ends where the day's work began and ended. As it dries this unbroken slab tends to contract and draw itself over the subgrade from each end toward the center. It is wholly or partially prevented from assuming its normal length, which becomes shorter and shorter, by the frictional resistance of the subgrade, and it is therefore subjected to a tensile stress which increases as it dries.

We may imagine the slab to consist of transverse segments each of which is contracting within itself and each tending to draw its neighbor toward it, and through its neighbor all others to the free end. Movement of each of these segments is resisted by subgrade friction at its base. When, therefore, the initial segment is pulled upon by its neighbor it is held back by the subgrade friction on its base. The second segment, pulled upon by the third, is also held back by forces of friction which are added to those of the first in resisting the movement of the two segments; and similarly the third, fourth, and succeeding segments, each pulls upon the train of segments behind it and is resisted by the combined forces of friction acting upon them.

The pull exerted by any segment is the stress in the concrete at the particular point, and it will be seen from the foregoing that this stress must increase with distance from a free end. It goes on increasing with distance until a point is reached at which it exceeds the tensile strength of the concrete, and at that point a crack occurs.

From the moment the crack forms a new condition is established. There has previously existed a condition of increasing stress from the free end toward the point of cracking. As the stress has increased the deformation has increased. We may imagine that the first segment at the free end has been able to contract practically to its normal length, its contraction being resisted only by its own subgrade friction. The second, however, has had to pull not only against the friction on its own base but against the frictional forces of its neighbor as well. It has, therefore, not been able to assume quite so nearly its normal length as the first; and the third, fourth, and succeeding segments have been increasingly deformed from their normal length. At the point of cracking the deformation is that which results from the tensile breaking stress of the concrete.

But the moment the crack forms this stress is relieved; a new free end is formed, and the concrete near this new end becomes able to assume more nearly its normal length, which it does, thereby opening the crack.

THE CRACKING OF REINFORCED CONCRETE PAVEMENTS

The action that has been described is what takes place when the pavement is not reinforced. If steel is used the action is similar up to the point of cracking, the only difference being that the steel adds its strength to the strength of the concrete in resisting the increasing stress and thus increases slightly the distance between the free end and the first crack in the concrete. But the steel does not crack. It remains as a connection between the dissevered slab and the balance of the pavement. It remains under stress and prevents the concrete in the dissevered slab from assuming its normal length. It keeps the concrete adjacent to the crack in a state of stress nearly equal to that which broke it, and holds the dissevered slab closely to the balance of the pavement, so that the crack in the concrete is not permitted to open. As near the crack the concrete remains in a high state of stress it may be assumed that secondary cracks will readily be formed near the initial crack as a result of bending stresses induced by the passage of vehicles.

Passing from the dissevered slab into the balance of the pavement it enters with an initial tensile stress, which, with respect to the balance of the pavement considered as a free body, becomes an external force which adds itself to the forces of friction resisting the contraction of the concrete and causes the next crack to occur in a lesser distance than the first.

The stress in the steel is increased as it passes through the second slab and continues to increase as it passes through successive slabs until finally the yield point is reached and the steel "necks down" or ruptures. At that point an open crack is formed. This behavior of reinforced concrete was first observed in the experiments of the Bureau of Public Roads on the Columbia Pike experimental road near Washington, D. C., and it has been described in a report published in this magazine.[1]

[1] Reinforcing and the subgrade as factors in the design of concrete pavements, by J. T. Pauls, PUBLIC ROADS, vol. 5, no. 8, October, 1924.

THE END OF THE CURING PERIOD A CRITICAL TIME

Whenever contraction cracks are formed in a pavement, whether it be in the early stage of its life or at a later priod, the behavior of the pavement prior to cracking is as described above. However, the critical period begins at the end of the curing. The pavement, which previously has been kept in an expanded condition, then begins to contract. The amount of the contraction as measured under laboratory conditions has been found to be as much as 0.0005 inch per inch of length, which is as much as would be caused by a change in temperature of 100° F. This contraction takes place in a period of at least three months under rapid drying conditions. In the field a longer period would undoubtedly be required. But this contraction occurs at a time when the tensile strength of the concrete is lower than at any subsequent period. For this reason contraction cracks are more likely to occur at this period than at any other time; and the strength of the concrete at this period is, therefore, one of the governing factors in any design which aims to prevent the formation of contraction cracks.

Unfortunately the tensile test for concrete is difficult to make and the results are quite variable. It is not possible, therefore, to arrive at a very exact determination of the tensile strength of the concrete during the early periods of its life. For practical purposes it is assumed that the tensile strength is equal to approximately one-twelfth of the compressive strength, and, although it must be granted that there will be considerable variation from them, the values of the tensile strength arrived at in this way are as follows:

Approximate tensile strength of concrete at various early stages

Age (days)	Tensile strength (pounds per square inch)	Age (days)	Tensile strength (pounds per square inch)
7	100	28	250
15	194	90	400

JOINTS AND REINFORCEMENT AS CRACK PREVENTIVES

The use of joints and steel reinforcement are the principal methods employed to prevent the formation of shrinkage cracks. If only the joints are used their effectiveness depends upon their being spaced at intervals slightly shorter than those at which the pavement would naturally crack. When steel is used the fine cracks which form are not generally regarded as objectionable, and the purpose of design may be limited to the prevention of the open cracks that form where the steel is "necked down" or ruptured. Obviously, therefore, the use of steel should be coupled with the use of joints at which the steel is separated, so spaced as to prevent the rupture of the steel and the formation of an open crack.

However, if the steel is entirely separated at the joints it is prevented at these points from acting as a dowel, which is one of its most valuable functions. If, on the other hand, it is not separated in some way it will rupture at intervals and will be no more effective as a dowel than if it is separated at joints. The only means thus far suggested to preserve continuous dowel action is that of greasing the steel, but this method prevents the bars from acting as temperature reinforcement and permits cracks to form as frequently and as wide as in plain concrete.

The method of design and construction which will now be proposed substitutes for the prevailing empirical methods a rational method of spacing the joints, permits the steel to act as temperature reinforcement between joints, and also preserves it valuable dowel action at the joints.

For the determination of the joint spacing it is necessary to know or assume the strength, dimensions, and other properties of the steel and concrete and the coefficient of friction between the pavement and the subgrade. The range of the latter coefficient was determined approximately by tests made by the Bureau of Public Roads several years ago and described in a recent article in PUBLIC ROADS.[2] These tests seem to show that the coefficient of friction varies between the approximate limits of 1.0 and 2.0. No doubt there is a possibility of its exceeding 2.0 under extremely rough subgrade conditions; but for purposes of design under normal conditions a value of 2.0 may be considered as safe and conservative.

THE RATIONAL SPACING OF JOINTS ILLUSTRATED

To illustrate the method employed, let it be assumed that the curing period has terminated at the end of 10 or 15 days. At this time the tensile strength of the concrete probably varies from 120 to 190 pounds per square inch, the probability being that the lower figure is more nearly correct. In that case a factor of safety of 4 would give a safe value of 30 pounds per square inch. If, then, it is desired to find the proper spacing of the joints, such that no cracking will occur between them, it is necessary to equate the total safe tensile resistance of the concrete and steel at the mid-section between the joints to the total frictional force between the mid section and a free end (see fig. 1). This condition is expressed by equation 1.

$$f\frac{L}{2}WB = 12\,BtS + a\frac{E_s}{E_c}S \qquad (1)$$

in which—

L = Spacing of transverse joints in feet.
B = Width of pavement in feet.
S = Allowable tension in concrete in pounds per square inch.
f = Coefficient of subgrade friction.
W = Weight of concrete in pounds per square foot.
t = Thickness of pavement in inches.
E_s = Modulus of elasticity of steel.
E_c = Modulus of elasticity of concrete.

The required spacing of the joints to satisfy the above condition when various amounts of steel are used is shown in the first column of Table 1. In these computations the values assumed for the various factors other than a are as follows:

S = 30 pounds per square inch.
$\frac{E_s}{E_c} = 10.$
$f = 2.0.$
W = 75 pounds.
t = 6 inches.
B = 18 feet.

It will be observed that the use of a rather large amount of steel is of very little value in increasing the spacing of the joints provided no crack is permissible between these joints.

[2] Friction tests of concrete on various subbases, by A. T. Goldbeck, PUBLIC ROAD vol. 5, no. 5, July, 1924.

If it is desired to find the spacing such that no open cracks will form, fine cracks being permissible, then it must be assumed that the total tension set up by the force of friction is resisted by the steel, and the joints must be so spaced that the steel will not be stressed to its yield point. This condition is satisfied by equation 2, as follows:

$$f\frac{L}{2}WB = a\,S_1 \qquad (2)$$

in which f, L, W, B, and a represent the same factors as in equation 1, and

S_1 = Allowable tension in steel in pounds per square inch.

Assuming values of f, W, and B as in the former example and the value of S_1 as 25,000 pounds per square inch the spacing for various amounts of steel is shown in column 2 of Table 1. From this column it will be seen that if fine cracks are permitted to develop the spacing of the joints may be materially increased by increasing the amount of steel.

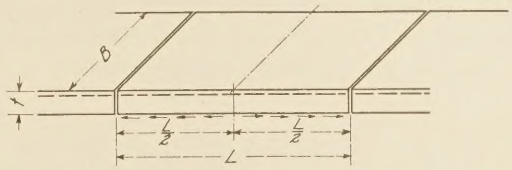


FIG. 1.—Diagram illustrating theory of crack spacing in reinforced concrete pavements

TABLE 1.—*Spacing of transverse joints to prevent (1) all intermediate cracking, and (2) intermediate wide cracks*

Amount of longitudinal steel	Required spacing of transverse joints	
	(1) For no intermediate cracks	(2) For no wide intermediate cracks
	Feet	*Feet*
Plain concrete	28.8	28.8
4¾-inch round bars (a=1.76 square inches)	29.2	32.6
8¾-inch round bars (a=3.52 square inches)	29.6	65.2
12¾-inch round bars (a=5.28 square inches)	30.0	97.8

THE DESIRABILITY OF SMOOTH SUBGRADES AND LONG CURING

It will be observed in equation 1 that L, the spacing of joints, varies directly as the tensile strength of the concrete and inversely as the coefficient of friction at the base. The equation emphasizes the importance of curing the concrete long enough to permit it to attain a relatively high tensile strength before it is allowed to shrink and thus develop tensile stresses. It also shows that the subgrade should be made as smooth as it is practicable to make it in order to keep the force of friction low and thus reduce the tension induced in the concrete when shrinkage takes place.

It is to be acknowledged that the coefficient of friction is the most uncertain of the factors involved in the calculations of the safe joint spacing recorded in Table 1. It is possible that the value of f, assumed to be 2.0 in these calculations, may under some conditions be much less. In that case the joint spacing would be much wider. It is probable that the assumed value of the coefficient is practically the maximum value and the error in the recorded spacing may therefore be regarded as on the safe side. It is possible also that the values assumed for other constants in the equations may be subject to revision as our knowledge increases. It is intended here to suggest a method rather than to determine exact values. It should be possible to obtain a working knowledge of the coefficient of friction that must exist on various subgrades by observation of the intervals at which contraction cracks form in existing pavements on the various soils.

DOWEL ACTION OF STEEL PRESERVED BY NEW FORM OF JOINT

Having determined the spacing of the joints there is the practical problem of how to form them. The method employed in determining the spacing under each of the two conditions examined assumes that a joint will be formed in the steel as well as in the concrete. This is accomplished by the proposed method without sacrifice of the valuable dowel action of the steel at the joints, as shown in Figure 2.

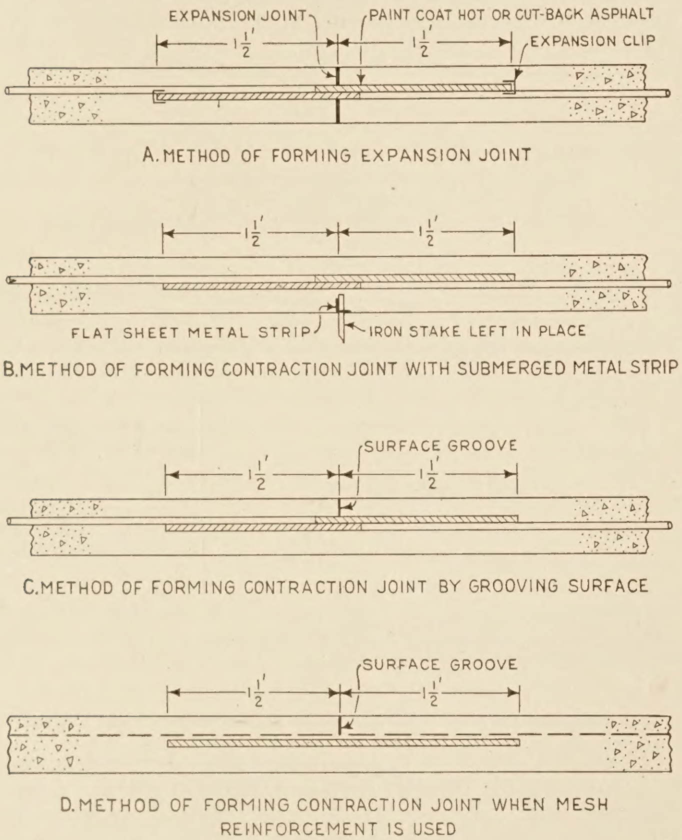


FIG. 2.—Methods proposed for forming joints in steel reinforcement and reinforced concrete pavements. The methods permit the contraction of the concrete without rupturing the steel

The desirable dowel action is provided by extending the steel from each slab a distance of several feet into the adjacent slab, and painting the extended ends of the bars with a hot asphaltic material to break the bond with the adjacent slab. If an expansion joint is required the method is as shown in Figure 2A.

However, it will probably not be necessary to provide expansion joints at the frequent intervals indicated in Table 1. At many of these intervals it will be necessary only to provide for contraction. This may be accomplished as shown in Figure 2B by introducing a submerged thin steel plate in such a way as to form a plane of cleavage. In this case no joint will be formed during construction, but a straight, wide crack will form over the submerged plate at the predetermined location. These cracks would need maintenance, but would probably be no more difficult to maintain than a formed joint. The same object might be accomplished by cutting a groove a few inches deep in the surface at the proper stage of hardening as shown in Figure 2C. Finally, if mesh, instead of bar reinforcement is used the same object is attained by the use of asphalted dowels as shown in Figure 2D.

By the use of any of these methods it is assured that the stress in the steel will only be that due to the subgrade friction on a predetermined length of the concrete, and the overlapping of the bars provides dowel action at the joints as well as at other points.

NEW MOTION PICTURE RELEASED

"Crossing the Great Salt Desert" is the title of the latest motion picture on road building which has been prepared for the Bureau of Public Roads by the Office of Motion Pictures, United States Department of Agriculture. The film, which is now ready for distribution, may be obtained by application to the Office of Motion Pictures, United States Department of Agriculture, Washington, D. C. Copies of the film are furnished free except for transportation charges both ways.

"Crossing the Great Salt Desert" is the story of a barrier overcome. It tells the story of the perils and privations of the early settlers who, with their wives and families, struggled in covered wagons across the Great Plains, fought off hostile Indians, traversed snow-covered mountain divides, and finally arrived at the eastern border of the Great Salt Desert.

Depending upon the season they then were forced either to stagger across this arid waste under the blistering rays of a summer sun or to plod wearily over the mud flats or brine-covered salt deposits in the winter. This desert was the terror of the western emigrants. Those who reached the other side in safety usually were successful in completing the westward journey and in reaching the promised land of settlement toward which all their arduous labors had been directed.

Animated views illustrate the northerly transcontinental road followed by the Forty-niners across the Great Salt Desert during the California gold rush. The southern route around and over the southern borders of the desert is shown to have been followed at a later date by Mormon settlers. The selection of one of the transcontinental routes, which forms a part of the Federal Aid Highway System, made necessary a choice between the old north and south roads. The northerly route was selected for the highway because it had the advantage in distance, travel time, snow conditions, grade, and ultimate cost.

The picture is replete with detailed construction views of the 40-mile section across the Great Salt Desert between Wendover and Knolls, Utah, known as the Wendover Cut-off. This was built as a Federal-aid project by the Utah State Highway Department in cooperation with the Bureau of Public Roads. The unique machinery used for making the embankment across the brine-covered salt beds, the difficulties of construction, the hardships of the men and teams, automobiles speeding over the completed roadway, a striking Utah sunset, and many other details, all make this picture of special interest to road builders and to the general public.

A NEW TEST FOR CONSISTENCY OF CONCRETE APPLICABLE TO DRY PAVING MIXTURES

DEVELOPED BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by F. H. JACKSON, Engineer of Tests, and GEORGE WERNER, Assistant Scientific Aid

THE development of a simple yet reliable method for determining the consistency of cement concrete mixes presents a problem which has never been satisfactorialy solved. Although numerous schemes have been proposed from time to time, only two or three of them have ever been developed so as to be of any practical value. The so-called slump test [1] has been used more extensively than any other and, when properly interpreted, has given fairly good results, especially as a method of field control. Its obvious limitations, however, especially as regards its use for the control of dry mixtures, lean mixtures, and concretes in which large-sized crushed stone is used, have prevented its general adoption. The flow test [2] is used to a certain extent as a method of laboratory control, but the nature of the apparatus required makes it impractical or use in the field. These are the more widely known methods. There is one other that merits serious consideration, namely, the penetration test for "workability" of concrete proposed by Pearson and Hitchcock;[3] and this method would appear to be limited in application to concrete of the consistency usually required in reinforced concrete building work and unsuited to the dry mixes employed in road construction.

There has been a great deal of discussion in recent years as to just what is meant by the terms "consistency," "workability," etc., as applied to concrete mixtures. According to Abrams,[4] consistency may be defined as "the relative plasticity or workability of freshly mixed material." Pearson and Hitchcock, on the other hand, in their paper describing the penetration test, distinguish between consistency and workability, holding that the former term should be used only to describe the condition of the concrete as it is affected by changes in water content, whereas the latter term should be used to describe that condition of a given mixture which depends not only upon the water content, but upon any factor which affects the amount of work required to place and finish the concrete in a satisfactory manner. It is, of course, apparent to anyone that there is a great difference between the "workability" of a rich, high-sanded, 1:2:3 mix and a lean, 1:3:6 mix, even though both mixes may have the same "consistency" as measured by the slump or flow test. Furthermore, it is impossible by any change in the water content of the leaner mixes to secure as workable a mix as may be obtained with the former. From this viewpoint, therefore, it would appear that there is justification for a distinction between the terms. It has often been demonstrated, however, that by far the greatest variation in the strength of concrete in actual practical construction is due to changes in the water content. Granting, therefore, the effect of water on strength, and bearing in mind also that variations in the amount of water required to produce concrete of a given consistency may be caused by any of the factors which affect workability, such as gradation of aggregate, cement content, etc., it would appear that the practical control of the water is the vital thing to be considered. In this paper a new test is described which may be used for controlling the water content and therefore the strength of any given concrete mixture.

The test was designed as a substitute for the slump test and is proposed for use primarily as a method of field control. The process is based upon the principle that the consistency of concrete may be determined by weighing the amount which is retained upon a circular plate of given diameter when the concrete is deposited thereon in any standard manner. The device which has been used by the Bureau of Public Roads in demonstrating this principle is shown in Figure 1.

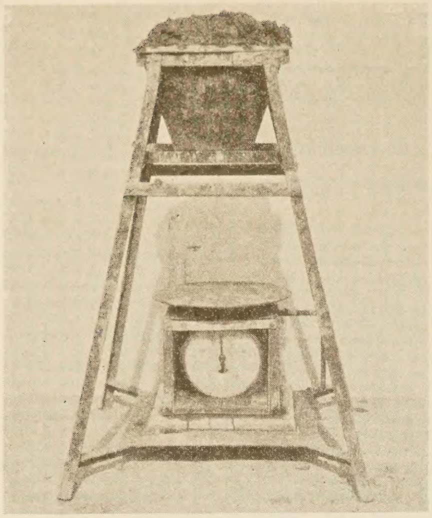
DESCRIPTION OF THE APPARATUS AND TEST

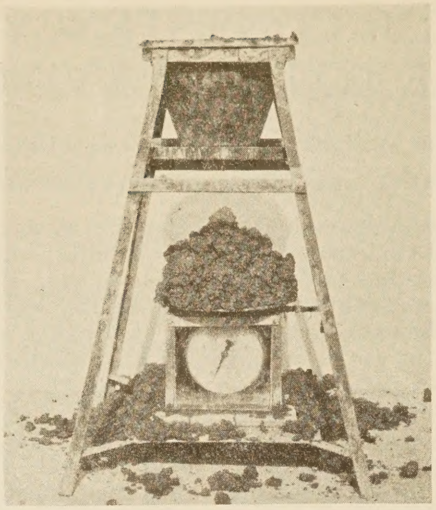
The apparatus consists essentially of a box spring scale upon which is mounted a circular steel plate 12 inches in diameter. A hopper supported by an angle-iron frame is mounted over the plate, so that the point of discharge is 12 inches from the surface of the steel plate. The hopper, which will hold approximately 45 pounds of wet concrete, is provided at the discharge end with a steel slide, the withdrawal of which permits the concrete to fall upon the plate. The test is made in the following manner. The apparatus is set up adjacent to the point where the batch of concrete to be tested will be deposited, and the hopper, which will hold about 50 per cent more concrete than it is possible to retain upon the 12-inch plate under any circumstances, is filled directly from the pile by means of shovels. It has been found that the exact procedure to be followed in filling the hopper is immaterial so long as it is filled completely and no attempt is made to jostle or compact the concrete. Mounted beneath the steel plate are two cams, operated by a handle, and so arranged that the weight on the plate may be taken off the spring. With the cams set in this position the slide is drawn and the concrete is allowed to flow out upon the plate until the hopper is empty, and when all movement has ceased the handle controlling the cam is turned and the concrete upon the plate is weighed. This weight is taken as the index of the consistency of the concrete. Views of the apparatus with the hopper full and after discharge of a wet and a dry concrete are shown in Figure 1.

THE RELIABILITY OF THE TEST

Numerous tests with this device on concrete of various proportions and with various sizes and types of aggregate indicate that it will truly measure the relative consistency of the mix. In one series of tests, for instance, in which gravel graded up to 1½ inches in size was used in a 1:2:4 mix, the amount of concrete retained upon the plate varied from 21 pounds with a water-cement ratio of 0.85 to 6 pounds with a water-cement ratio of 1.2. In this series of tests which was made some months ago a number of batches of 1:2:4

[1] U. S. Dept. of Agric. Bul. 949, Standard and Tentative Methods of Sampling and Testing Highway Materials, p. 66.
[2] Proc. of the American Society for Testing Materials, vol. 21, p. 983.
[3] Proc. of the American Society for Testing Materials, vol. 23, pt. 2, p. 276.
[4] Proc. of the American Society for Testing Materials, vol. 23, pt. 2, p. 443.





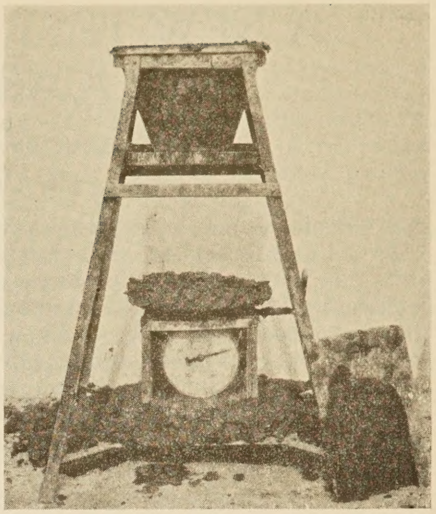


FIG. 1.—Left—the device with hopper full. Center—after discharge of a dry concrete. Right—after discharge of a wet concrete

concrete were made in which the water-cement ratio was increased by increments of 0.05. Slump tests, flow tests, and tests with the 12-inch plate were made on each batch after which the material was cast into 6 by 12-inch cylinders and tested for compressive strength at the age of 28 days. Results of this series of tests are shown in Figure 2. Note the striking similarity between the curve showing the relation between water-cement ratio and crushing strength and the curve for water-cement ratio and consistency as determined by the plate tester. It will be observed that there is a tendency for the amount of concrete retained upon the plate to drop off for very dry consistencies in exactly the same way as the strength falls off. It occured to the authors that if the relationship shown in Figure 2 were true throughout the entire range of working consistencies and for various mixes and types of aggregate that the plate test should give directly a very good indication of the probable relative strength of the concrete. In this particular series the slump test showed up very poorly, although it was made strictly in accordance with the methods described in the A. S. T. M. tentative standards, except that the mold was removed immediately after placing the concrete. It will be noted that the concrete did not slump at all until a water-cement ratio of 1.1 had been reached, corresponding to a flow of 160 and a plate test of 9 pounds. Granted that this is very unusual, these results nevertheless serve to illustrate the uncertainties of the slump test. The flow test results in this series were, on the other hand, very concordant, showing practically a straight line relation for a water-cement ratio varying from 0.75 to 1.2.

Having in mind the interesting possibilities in the way of strength control indicated by this initial series of tests, a somewhat more extensive investigation was begun for the purpose of determining whether these relations held for different types and sizes of aggregates and different proportions. In this series of tests, six different mixes were used: 1:1½:3, 1:2:3, 1:2:3½, 1:2:4, 1:2½:5, and a 1:3:6. Three types of aggregates—Potomac River gravel, crushed limestone, and crushed slag—were employed for each mix. The gravel and slag were graded from one-fourth inch to 1½ inches and the limestone from one-fourth inch to 1½ inches for one lot and one-fourth inch to 2½ inches for another lot. The concrete was mixed by hand with shovels in 1 cubic foot batches; and four 6 by 12 inch compression specimens were made from the concrete in each batch. Three consistencies were used for each combination—dry, medium and

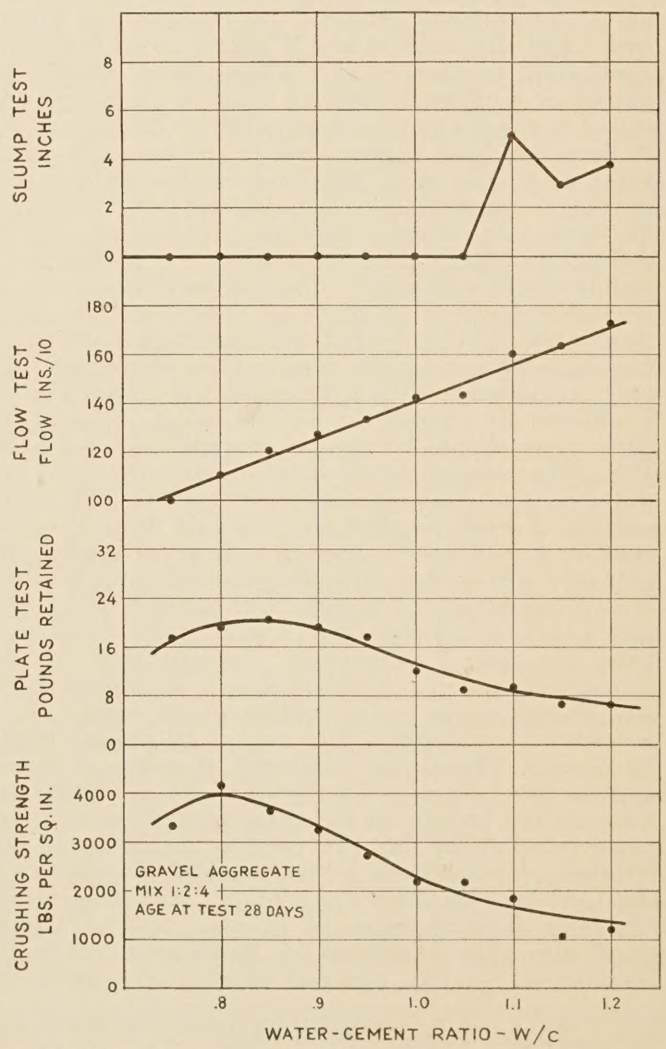


FIG. 2.—Relation of water-cement ratio and strength and corresponding consistency relation as determined by the slump, flow and plate tests

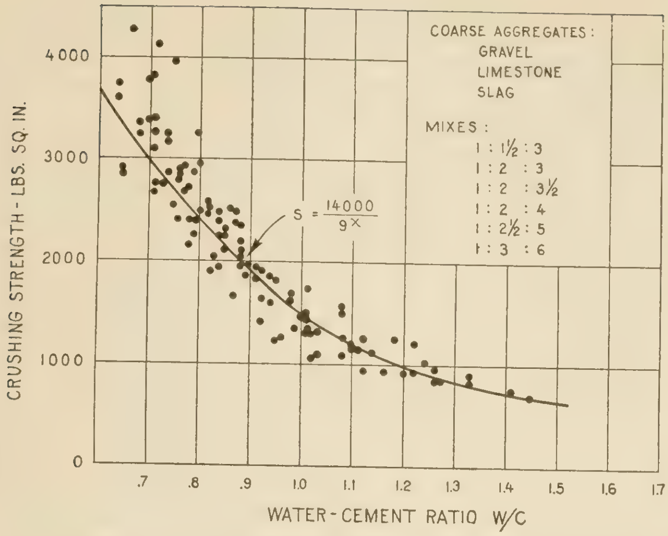


FIG. 3.—Relation of crushing strength and water-cement ratio for all compression tests, showing comparison with the curve suggested by Abrams and Walker

wet: and immediately after mixing, the three tests for consistency—plate test, flow test, and slump test—were made simultaneously on different portions of the batch, by different operators, in order to eliminate any error due to the time element or to rehandling the concrete. Finally, at the conclusion of the first series the entire program was repeated, making a total of approximately 130 separate batches of concrete tested.

COMPARISON WITH OTHER TESTS

In order to check the accuracy of the strength-water-cement-ratio relation, the results of all the compression tests, irrespective of mix, size, or type of aggregate were plotted, and are shown in Figure 3, together with the curve suggested by Abrams and Walker[5] as indicating the strength-water-cement-ratio relation. The equation of this curve is $S = \frac{14{,}000}{9^x}$, in which S equals the crushing strength of the concrete in pounds per square inch at 28 days, and x (an exponent) is the water-cement ratio. It will be seen that the average of all points lie very close to the curve except for the very dry mixes, although it should be noted that these tests were made at the age of 14 instead of 28 days, so that the average of all of the points would probably lie somewhat higher than is here indicated if they had been made at the conventional period.

To determine the relations between the strength and the slump, flow, and plate tests for the different mixes and types of aggregate, the results were plotted separately on a series of charts which are reproduced as Figure 4. It will be observed that the relation between strength and consistency as determined by the plate test is the same, in general, as that shown in Figure 2. Furthermore, the points representing consistency of concrete made with the different coarse aggregates appear with one or two exceptions to lie very much

[5] Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill., Bul. 9, 2d Ed., April, 1925.

(Continued on page 136)

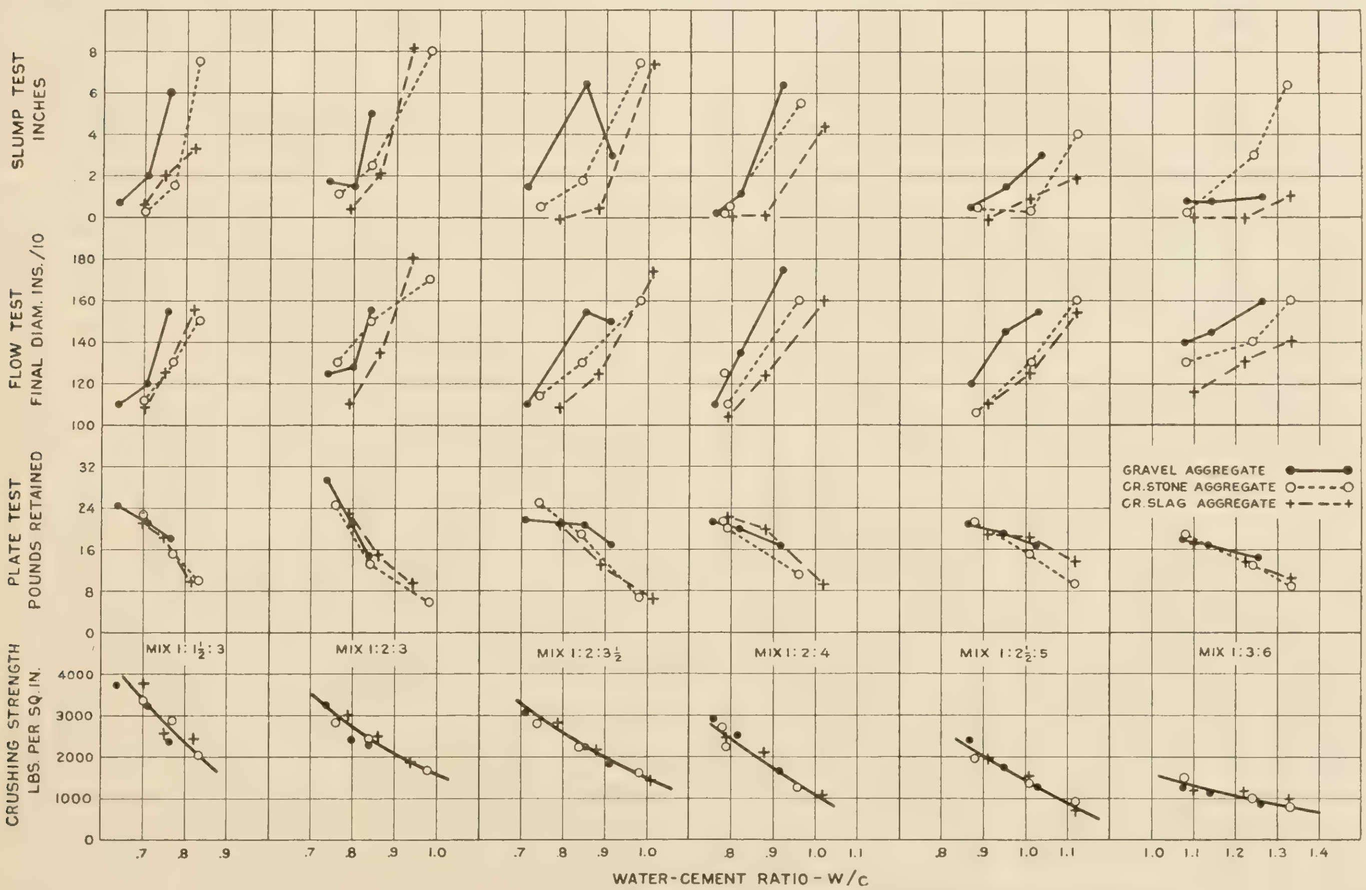


FIG. 4.—Results of strength tests and consistency tests made by the slump, flow, and plate methods on gravel concrete

COMMODITY TRANSPORTATION BY MOTOR TRUCK

A SUMMARY OF THE RESULTS OF TRANSPORTATION SURVEYS CONDUCTED BY THE U. S. BUREAU OF PUBLIC ROADS

Reported by J. G. McKAY, Chief, Division of Highway Economics

THE rapid growth of motor-truck transportation during the past few years has resulted from five principal causes, which are: (1) The relative economy of motor-truck transportation with respect to transportation by horse-drawn vehicles; (2) the development of improved rural highways; (3) the demands of shippers for prompt service; (4) the constant improvement in the mechanical efficiency of the vehicle; and (5) the impetus given to motor-truck transportation by the World War.

Mainly as a result of the operation of these causes the registration of trucks in the United States increased from 136,000 in 1915 to 2,130,000 in 1924, an increase of over 1,400 per cent. Although the truck traffic on rural highways has apparently increased at a somewhat slower rate than the registration, it has already attained a volume which makes it a very important part of our system of distribution. As an illustration of the relative growth of traffic and registration it has been found that in Maine motor-truck traffic increased approximately 500 per cent from 1916 to 1924, while the registration increased approximately 850 per cent. As an indication of the magnitude of this modern development it may be noted that the transportation of commodities over the Connecticut highway system during the year beginning in September, 1922, reached a total of 88,000,000 ton-miles, and that in Cook County, Ill., during the summer months of 1924 the average daily movement was 61,000 net ton-miles.

The motor trucks used on rural highways are predominantly trucks of small capacity. Results of the highway transportation surveys in the States of Maine, Pennsylvania, and Connecticut, and in Cook County, Ill., indicate that trucks of ½ to 2½ tons capacity constitute from 77.3 per cent in Cook County to 96.8 per cent in Maine of the total number of trucks operating on the rural highways. Trucks of 5 tons capacity, or greater, constitute from 0.3 per cent in Maine to 11.7 per cent in Connecticut of the total number found on the rural highways. Near large cities more heavy-duty trucks operate on the highways, but the truck of large capacity does not travel far from centers of population in the movement of goods.

TRUCK HAULAGE PREDOMINANTLY A LOCAL MOVEMENT IN ALL AREAS

The bulk of the motor-truck movement is local, short-haul transportation. There is some variation in the length of haul depending upon: (1) The prevailing production of the area; (2) shipping distances to market; (3) the type of highway improvement; (4) the other types of transportation available; and (5) the distance between centers of population. The comparison of truck haulage in Connecticut, Maine, California, and Cook County, Ill., to be found in Table 1 indicates clearly the predominance of the short-haul movement.

The principal movement of loaded trucks occurs within a zone of 29 miles or less in Connecticut, California, Maine, and Cook County. In Connecticut 79.5 per cent, in California 60.7 per cent, in Maine 80.5 per cent, and in Cook County, Ill., 75.8 per cent of the loaded trucks move less than 30 miles, while in the latter case 55.3 per cent operate less than 20 miles. In Connecticut and Maine 47 per cent move less than 10 miles. Only a small percentage of the truck movement exceeds 60 miles; in Connecticut 7.9 per cent, Maine 6.6 per cent, and in Cook County, Ill., 5.4 per cent.

TABLE 1.—*The percentage of loaded trucks operating various distances in several areas*

Length of haul (miles)	Percentage of loaded trucks, Connecticut, 1922–1923	Percentage of loaded trucks, California, 1922 [1]	Percentage of loaded trucks, Maine, 1924	Percentage of loaded trucks, Cook County, Ill., 1924
	Per cent	*Per cent*	*Per cent*	*Per cent*
0–9	47.7	26.8	47.0	24.3
10–19	21.3	22.2	23.1	31.0
20–29	10.5	11.7	10.4	20.5
30–39	6.9	8.3	6.9	9.4
40–49	2.9	5.5	4.1	6.6
50–59	2.8	6.5	1.9	2.8
60–69	1.3	4.7	1.2	1.1
70–79	1.5		.8	.6
80–89	.5	3.1	.7	.4
90–99	.5		.5	.5
100 and over	4.1	11.2	3.4	2.8

[1] California data limited to important highways upon which the percentage of long-haul trucking is greater than upon all highways in the State.

A more exact analysis, based on the net tonnage hauled in the various mileage zones in Connecticut, is presented in Table 2.

TABLE 2.—*Percentage of the total net tonnage hauled in Connecticut by mileage zones*

	Mileage zones											
	0–9	10–19	20–29	30–39	40–49	50–59	60–69	70–79	80–89	90–99	100 and over	Total
	Per cent of total net tonnage											
Products of agriculture	2.8	1.7	0.9	0.5	0.2	0.4	0.1	0.2	0.0	0.1	0.3	7.2
Products of animals	2.9	2.1	1.3	1.0	.6	.4	.2	.4	.1	.1	.3	9.4
Products of mines	7.5	1.4	.2	.2	.1	.1	.1	.0	.0	.0	.0	9.6
Products of forests	2.3	.8	.6	.3	.1	.1	.1	.0	.1	.0	.4	4.8
Products of manufactures	20.8	13.2	8.6	7.1	3.0	3.6	1.7	2.0	.6	.8	7.6	69.0
Total	36.3	19.2	11.6	9.1	4.0	4.6	2.2	2.6	.8	1.0	8.6	100.0

Of Connecticut's total net tonnage, as shown by the table, 67.1 per cent is hauled 29 miles or less and 15.2 per cent is hauled 60 miles or more. Household goods constitute a considerable portion of the goods hauled long distances in this State. Of the total tonnage, 7.2 per cent is composed of products of agriculture, 9.4 per cent products of animals, 9.6 per cent products of mines, 4.8 per cent products of forests, and 69.0 per cent are manufactured products. Manufactured goods are hauled over longer distances than goods of the other principal commodity groups, the large percentage reflecting the type of production in the State. The movement of products of mines is restricted largely to distances less than 10 miles and is primarily a movement of sand, gravel, stone and coal.

THE MOTOR-TRUCK MOVEMENT LARGELY A DISTRIBUTING SERVICE

A large percentage of motor-truck transportation is a direct distribution of commodities to points of final use. In Cook County, Ill., approximately 70 per cent of all loaded trucks were found to be engaged in a service of this character. Analyzed according to destination 25.8 per cent of the loaded trucks were found to be hauling to retail establishments, 17.7 per cent to consumers, 20.8 per cent to construction and repair jobs, 5.5 per cent to farms, and 30.2 per cent to other destinations. Of these several groups all but the last mentioned were engaged in a service of direct distribution of consumption goods or in delivering goods to the point of final use.

Food products and construction materials are the two principal types of commodities hauled by motor truck. In Cook County the analysis of commodities hauled resulted in the following classification:

Commodity	Per cent of net tonnage
Fruits and vegetables	8.0
Miscellaneous products of agriculture	1.4
Milk and cream	4.8
Other dairy products	.4
Meats and packing-house products	1.7
Miscellaneous products of animals	2.4
Bakery goods	1.4
Groceries	4.9
All other commodities	75.0
	100.0

As will be seen from the foregoing, the principal food products make up 25 per cent of the net tonnage. Miscellaneous food products, which are not individually important but which in the aggregate constitute a considerable tonnage, will increase this percentage. Approximately 32 per cent of the daily milk supply of Chicago is brought into the city by motor truck, and the greater part of the milk shipped to the city from points within a radius of 50 miles is trucked in.

Commodities hauled by truck in a given area reflect largely the production in the area, the size and distribution of centers of population and to a limited extent the adequacy of the service of other means of transportation available for the movement of commodities.

COMMERCIAL TRUCK HAULING KEENLY COMPETITIVE

The surveys made in the several areas show definitely that the bulk of the truck movement consists of the transportation of goods in trucks belonging to the owner of the goods. The strictly commercial trucker hauls but a small percentage of the total tonnage on rural highways, and this condition is likely to continue.

There is keen competition between the commercial motor truckers. This has resulted in losses in some cases and in a small margin of profit in others, and has caused a rapid turnover of the companies. A knowledge of operating costs is needed to measure the efficiency of any business, yet the commercial trucking companies know very little of their costs. A few of the larger ones with the better organization do keep cost records, but these records as well as the analysis and application of them as a control in management can be considerably improved. The creation of a depreciation fund to replace worn-out equipment is practically unknown to the smaller companies.

There is a distinct field for the commercial trucker; but successful operation and reliable service will depend largely upon:

1. The keeping of reasonably accurate records of the cost of performing service as a guide to operation.
2. Reasonable regulation to protect responsible operators from unfair competition.

As a result of the application of these two principles the following benefits will result:

1. Analysis of cost records will determine profitable and unprofitable business.
2. Commercial haulage rates can be accurately made.
3. The most economical type of transportation in the various mileage zones can be determined.
4. The proper operating equipment and size of equipment can be selected to serve the needs of the communities.
5. The business will be made more stable and secure, which will be of material advantage to users, and operators will be able to perform more satisfactory and efficient service and earn a fair return on their investment.

CLASSES OF MOTOR TRUCK TRANSPORTATION

The general field of motor-truck transportation can be divided into three major parts:

1. Local distribution of commodities.
2. Organized motor truck transportation supplementing existing rail and water service.
3. Long-haul truck transportation of special commodities.

Local distribution of commodities.—This class of transportation constitutes the bulk of the motor-truck tonnage movement. This movement is not strictly competitive with rail or water service. It consists primarily of the distribution of goods within cities and their suburban areas, and between cities and tributary areas.

There are two distinct types of commodity distribution: (1) The distribution of goods having both origin and destination within the local area, such as the wholesale to retail movement, retail to consumer, distribution of building materials, and the delivery of milk, garden truck, fruits and vegetables from rural areas to the cities; and (2) the completion by truck of the transportation service provided by rail and water facilities.

Local distribution will continue to be the bulk of motor-truck transportation and will be handled largely by the owners of the goods themselves, a small portion by the commercial trucker. In either case efficient distribution requires that accurate cost records be maintained. The merchant with an efficient distribution system has a distinct advantage in competition; the commercial trucker who knows his service costs can distinguish between profitable and unprofitable business and raise his profits accordingly. Operating without knowledge of the cost of doing business he can not determine unprofitable business and is usually a marginal or submarginal operator.

A more efficient use of terminal facilities requires close coordination of motor-truck transportation with rail and water facilities. A well-organized motor trucking company, operating in cooperation and preferably controlled by the major rail carriers in the area, would probably be the most efficient operating organization. Ownership of the terminal motor trucking company is not essential, but control by the rail and water lines is essential to guarantee service and becomes imperative if through rates for combined rail, water, and truck transportation are established.

As a major field for the development of motor-truck transportation this movement is limited to large centers of population, primarily the terminal areas.

Service supplementary to rail transportation.—Organized motor-truck service supplementing existing rail transportation offers three types of service.

1. The extension of freight service into areas without rail service.

2. The substitution of motor-truck service for rail service on unprofitable branch rail lines.

3. The combination of rail and truck service for short-haul package freight in terminal areas and on congested trunk lines. The development of this service is illustrated by the Pennsylvania and New York Central truck haulage of package freight. Prompt and rapid service and partial elimination of way-freight trains on heavy-traffic sections is largely responsible for this development. The factor of increased safety of train operation by the partial elimination of the package-freight train is a real advantage.

There is a real service to be performed by extension of motor-truck lines into areas without rail facilities. In such areas the development of motor-truck lines is perhaps the only type of modern transportation capable of performing service since it is becoming more difficult and hazardous to obtain capital for the extension of rail lines into new areas.

The substitution of motor-truck service for rail service on unprofitable branch lines represents economy in providing transportation service as a whole.

The combination truck-and-rail service for short-haul package freight on rail trunk lines demands a close coordination of service. Haulage of rail package freight by motor trucking companies on a contractual basis represents the first step in this development. If this new type of service is economically sound the carrier ultimately should own and operate the motor-truck equipment. Until it demonstrates its place in the scheme of transportation it is natural for the rail lines to shift the risk of ownership of equipment and a large share of operating risks to motor trucking companies. It is to be remembered also that the possibilities of growth of combined rail-and-truck transportation are largely limited to the comparatively few congested terminal areas and the heavy-traffic rail mileage. Its development depends largely upon the legalization of through rates to include the motor truck pick-up-and-delivery service.

The competition of motor trucking companies with rail lines is not economical, assuming that rail operation offers satisfactory service.

Long-haul transportation of special commodities.—This class of service by motor truck is economically sound when speed of delivery, or the avoidance of special packing and crating are the primary considerations. It is also justified for the transportation of perishable commodities. As for the balance of the long-haul movement it is very questionable, assuming efficient rail service, that it is economically sound, or that it is profitable to the motor-trucking company specializing in the long-haul field. Maintenance of cost records and a fair charge for the right to perform this service will determine whether or not this movement is economically sound. Perhaps the principal value of the long-haul trucker as a competitor or potential competitor has been to call the attention of rail management to the shippers' need for rapid and efficient handling of package freight.

Regulation of commercial trucking organizations is essential because of the disastrous effect of unregulated and destructive competition between motor trucking companies in a field in which the demand for service is relatively inelastic and the supply of service is elastic.

Interstate trucking is largely a local distribution affected by nearness to State lines. The balance of the interstate movement consists of long-distance haulage and is a very small part of the total. Regulation of interstate motor truck transportation means practically the regulation of local trucking and is therefore largely a problem of State regulation.

Jurisdictional disputes concerning the regulation of motor trucks and busses operating across State lines have already developed between the State agencies responsible for the regulation of this new transportation business. A national advisory body to adjust problems of regulation between the States may be of value. It is difficult to visualize the administrative ability of a national regulatory body capable of satisfactorily handling the multitude of cases that would naturally develop if complete control and regulation is vested solely in a national agency. If authority to control this new transportation agency is vested in a national agency, delegation of authority by the national control agency to State regulatory agencies, with power to make decisions, subject to final review on appeal to the national agency would possibly be of real service.

UNIFORM ROAD SIGNS ADOPTED

The Joint Board on Interstate Highways meeting in Washington, D. C., Aug. 3–4, adopted a system of interstate roads and a series of standard danger, caution, direction, and informational signs which it will recommend for use in marking and signing the systems selected.

The system adopted will now be mapped, and as promptly as possible a full report will be made to the Secretary of Agriculture, by whom the board was appointed last February.

The selected routes will be numbered in accordance with a system to be evolved by a subcommittee of the board.

The standard route marker will be a typical United States shield painted white, on which will appear in black the name of the appropriate State, the initials U. S., and the route number. If possible the route numbers will be limited to two digits for easy reading, and steps will be taken to prevent the use of the standard marker for any purpose other than for the marking of the selected system of interstate roads.

Other standard signs for grade crossings, curves, steep grades, etc., were adopted and full details as to design, character, and size of lettering and color worked out.

TAR PAPER ON LOESS SUBGRADE LESSENS HAIR CRACKS IN CONCRETE PAVEMENT

By R. W. CRUM, Engineer of Materials and Tests, Iowa State Highway Commission

HAIR cracks developed during the curing period on the first cement concrete pavement laid on the loess soil in the western part of the State by the Iowa State Highway Commission. This pavement extended through the hilly country adjacent to the flood plain of the Missouri River. The cracks or checks were of the kind usually called hair cracks, although they were generally longer and wider than those commonly encountered in patches on many concrete pavements. They were not surface cracks only. On the contrary, they extended through the pavement to the bottom of the slab.

FIG. 1.—Curved formation of hair cracks which is the typical condition when concrete pavement is laid downhill

Careful investigation of the construction conditions, together with some corroborative experiments, indicated that the cracking occurred when the loess soil was dry or slightly moist and most frequently when the pavement was being laid downhill. Upon later work this cracking was eliminated to some extent by thoroughly wetting the subgrade immediately prior to pouring the concrete. A much more effective method, however, was found to be the placing of a layer of tar paper on the subgrade before laying the pavement. This tar-paper treatment has been found to be especially successful and is now the standard practice for concrete pavements to be laid on this type of Iowa soil.

The use of tar paper is not suggested as a cure for all hair cracking. Other conditions than the character of the soil are doubtless often responsible. Apparently the tar paper does not decrease the amount of transverse and longitudinal cracking which develops after the pavement is built and opened to traffic, but definite conclusions can not be given concerning this phase of the matter because it has not been studied in detail.

DESCRIPTION OF HAIR CRACKING ON FIRST CONCRETE PAVEMENT

The data in Figure 3 describe the conditions under which this first concrete pavement was laid and the character of the cracking which developed. The significant conditions upon which the hair cracking depends may be observed on this typical section of the road. The subgrade was uniformly loess soil, hard and smooth, which had been lightly sprinkled just before the concrete was placed. Two brands of cement were used and changed at frequent intervals but this had no apparent effect upon the cracking.

Beginning at station 578 at the right of Figure 3 and proceeding toward the left, in the order that the concrete was placed, it will be noticed that for about 200 feet a large number of cracks appear and this is where the concrete was placed downhill (see fig. 1). Across the sag a few cracks may also be seen, but going up the other side there is none until the crest is reached. From there on, down to the foot of the grade, cracks are prevalent, but again across the bottom and up the hill on the other side only a few are found. Once more, as soon as the crest is passed they appear again until the close of work on August 20 at about station 523.

August 21, being Sunday, no work was done, and on August 22 a heavy rain occurred so that when work was resumed on August 23 the subgrade was in a thoroughly saturated condition. Even though the operation was still downgrade, no hair cracks appeared in the concrete laid immediately after the rain. But they appear again in that which was laid several days later when the subgrade had dried out. These phenomena were repeated several times in the 17 miles of pavement placed that year. Only three remedies suggested themselves—either the concrete would have to be always laid uphill (which, of course, is impracticable); or the subgrade would have to be saturated; or possibly an impervious layer might be placed between the subgrade and the concrete.

FIG. 2.—Location of the road over the loess formation and the steep angle of repose of this type of soil

CHARACTER OF THE LOESS SOIL

According to most authorities on Iowa geology the loess soil was deposited by wind over glacial drift. It extends back from the Missouri River for a distance of 50 miles or more and forms a mantle so thick that few highway cuts are deep enough to penetrate it. It is a very finely divided material which is eroded rapidly by running water, yet it will stand on a slope so steep as to be almost vertical. In recognition of these conditions it has become the standard practice

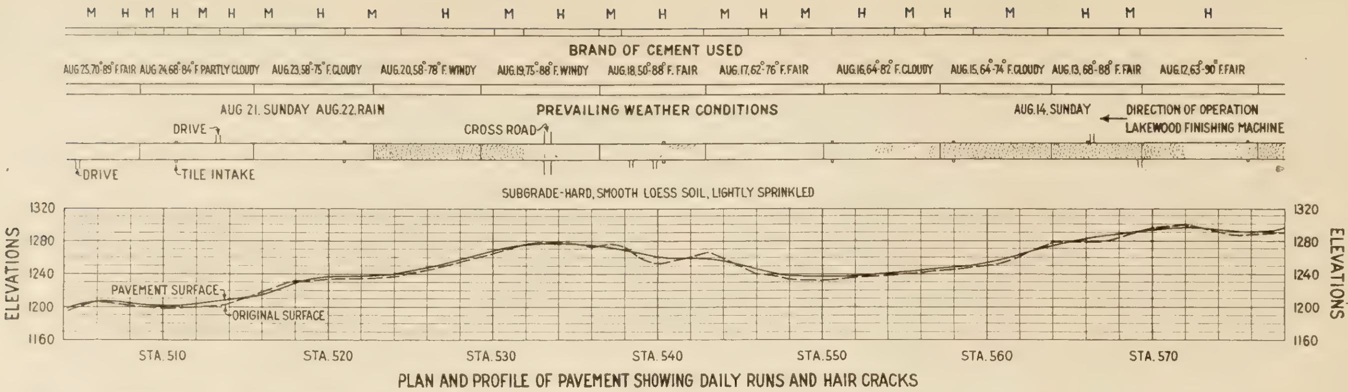


FIG. 3.—Typical section of first concrete pavement laid on loess soil of western Iowa. Hair cracking is most prevalent on portion of the pavement laid downhill

to excavate cuts to a practically vertical slope (fig. 2) and to carry surface water over the pavement, which is built with a curbed section, rather than in side ditches (fig. 4).

FIG. 4.—Topography of loess soil area and curbed section of concrete pavement

Typical physical characteristics of the soil, determined by the methods of test described in the April, 1925, issue of PUBLIC ROADS are as follows:

		Per cent
Mechanical analysis	Sand	0. 74
	Silt	67. 08
	Clay	32. 10
Moisture equivalent		21. 9
Capillary moisture		43. 7
Lineal shrinkage (by test of 1 by 1 by 10 inch bar)		2. 30

LABORATORY TESTS INDICATE MOISTURE ABSORPTION AS CAUSE OF CRACKING

Some corroborative studies were made in the laboratory to determine the cause of the hair cracks in the pavement laid on the loess soil. The studies were made on mortar slabs, 18 inches wide and 2 inches thick, the proportions being 1 part cement to 2.17 parts of sand by weight.

In these experiments the following phenomena were noted:

1. Slabs on dry or damp loess or yellow clay cracked badly within three hours after being placed.
2. Slabs on saturated clay or loess did not crack.
3. Slabs laid on tar paper did not crack.
4. Dry mixtures showed much less tendency to crack than wet mixtures.

PRESENT IOWA PRACTICE

In view of the foregoing observations, on the next contract the contractor was given the option of using tar paper or thoroughly wetting the subgrade for the length of one day's run ahead of the mixer. He chose the additional wetting, and there was decided improvement, although numerous hair cracks still developed. This method required a great deal of water, and when the subgrade was soaked enough to insure against the cracking it was really too muddy for the trucks to operate properly.

On later contracts the tar paper has been required and the troublesome hair cracks have been practically eliminated. On 14 miles completed in 1924 in a location similar in every way to that shown in Figure 3 only two small patches of cracks have been noted, and it is known definitely in one of these cases that the tar paper was omitted.

HIGHWAY RESEARCH BOARD TO REPORT ON REINFORCED CONCRETE ROADS

A report presented at a recent meeting of the executive committee of the Highway Research Board by C. A. Hogentogler, chairman of the special investigation of the economic value of steel reinforcement in concrete roads, shows that inspections have been completed on 375 miles of plain and reinforced concrete roads, varying in age from 1 to 10 years, and containing approximately 300 comparisons of slabs with and without steel which were subjected to the same influencing conditions. These roads are located in Massachusetts, Connecticut, New York, New Jersey, Pennsylvania, Delaware, Maryland, Virginia, North Carolina, Georgia, and Ohio. The States still listed for inspection are Michigan, Illinois, Missouri, Wisconsin, Iowa, Utah, Washington, California, Texas, and Mississippi.

It is thus assured that a sufficient number of direct comparisons will be made to warrant drawing definite conclusions regarding the effect of steel as influenced by age, design, traffic, climate, and subgrade, as well as by the type, weight, and placement of the reinforcement itself. Maintenance costs of both plain and reinforced concrete roads have also been secured on a large mileage. These costs are in such detail as to show how the maintenance is influenced by any desired variable.

The final report on this investigation will be one of the most complete studies of concrete roads ever undertaken. It will be presented at the fifth annual meeting of the Highway Research Board to be held in Washington, D. C., on December 3 and 4, 1925.

PRESENT STATUS OF THE TRUCK TIRE TESTS OF THE BUREAU OF PUBLIC ROADS

A PRELIMINARY REPORT

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by **E. B. SMITH**, Engineer of Tests

DURING the past two years the Bureau of Public Roads has been carrying on a series of tests to determine the relative cushioning properties of solid rubber, cushion, and pneumatic truck tires. This information is of vital interest to truck manufacturers because it may suggest a method for reducing the force of the shocks transmitted to the mechanism and thereby lengthen the life of the truck. The truck user is also benefited by any improvement which tends to decrease the annual operating costs of the truck. Truck manufacturers and users, however, representing only a small fraction of the population, are not the only beneficiaries, since the tests also provide data relative to the destructive effect of the different types of tires on the road surfaces which are built at the expense of the public.

WHEN AND BY WHOM TESTS WERE INITIATED

The tests were initiated about two years ago by representatives of the Bureau of Public Roads in conference with representatives of the various tire and truck manufacturers. A cooperative committee was chosen to prepare a preliminary outline of a series of tests which would furnish useful information relative to tires, trucks, and road surfaces. This cooperative committee represented the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America. The rather extensive series of tests which were outlined included the use of different trucks, most of the representative tires of different types and sizes (necessitating several hundred tire changes), and the constant employment of 8 to 10 men. It was expected that the series would extend over a period of more than three years.

The Bureau of Public Roads is conducting these tests at the Arlington Experiment Station, near Washington, D. C., and is providing all the personnel, testing facilities, apparatus, and instruments. The other cooperating agencies are furnishing all the tires, extra wheels, and trucks. After the tests were begun many unforeseen duplications were eliminated and the time of the investigation was considerably reduced. Present estimates indicate that this series of tests, which has been in progress less than two years, will be completed probably next fall, at least in so far as the experimental work is concerned.

In stating the facts to be determined by these investigations no one object can be selected as the most important. The principal factors involved are of course the cushioning qualities of various types of tires and the relative impact they deliver to road surfaces. However, these primary factors are influenced by variations in the sprung and unsprung loads, the speed, the temperature, the condition and kind of tire, and the roughness of the road surface. These variations necessitate principally tests under impact conditinos, but for each tire a static-load test is also made. In securing the data sufficient information is also obtained to determine the force delivered to road surfaces by both impact and static loads. It is possible that a careful compilation of the data will reveal a short and accurate test for classifying tires.

Although these tests are not finished, some of the preliminary indications have been already of real service to the automotive industry and improvements have resulted. Furthermore, the series is developing information which will be of value in the formulation of traffic rules and regulations.

TREND OF RESULTS

At this time it is not proper to publish any of the detailed numerical data which have been obtained, although it should be of interest, in a general way, to indicate the trend of the results.

It is found that the magnitude of the deceleration of a truck wheel may be as high as 2,500 feet per second, as in the case of a badly worn tire. In some cases this may produce an impact force equivalent to 15 times the static load. This, however, is a very extreme condition rarely found in actual practice. It corresponds to a truck carrying a load of 5 tons and with little or no rubber on the steel wheel rim.

Wide tires give higher impact forces than narrow tires for the same load. This does not mean that narrow tires should be used in preference to wide tires in order to prevent the destruction of road surfaces. On the contrary, tires should not be reduced in width beyond a load of 800 pounds per inch of width for wide tires and 600 pounds per inch of width for smaller tires (less than 5 inches in width) on account of the possibility of causing ruts in rock and asphalt roads and the further possibility of reducing the mileage service of the tire.

The vertical thickness of rubber available for cushioning in solid and cushion tires is a very important factor. An ordinarily worn tire may deliver an impact blow as great as seven times the static load of the same tire.

An unevenly worn or damaged tire may easily produce a force equivalent to that of a very rough road both in the impact the road surface will be subjected to and the equal and opposite shocks which will be transmitted to the truck.

On smooth pavements it is self-evident that the cushioning qualities of tires and the impact forces developed by traffic are of small value and consequence and not capable of being measured. Only on rough and wavy pavements do these qualities and force factors become of sufficient magnitude to be measured.

BRIEF DESCRIPTION OF IMPACT TESTS

The impact test consists in operating a truck equipped with test tires over representative sections of smooth concrete and rough granite-block roads, and over a series of artificial obstructions securely anchored to another section of smooth concrete road.

These artificial obstructions consist of inclined planes, rectangular and rounded blocks of varying heights. The impact is determined both on striking these obstructions and at the instant the wheel hits the ground on the first rebound. Sufficient variations in speed are made to determine a reliable curve and the truck load is varied from a condition of emptiness to such an amount that the tires will be overloaded 50 per cent. The determination of the total impact force is made by measuring the deceleration of a rear wheel and the deflection of the corresponding truck spring at the instant of impact.

The deceleration of the rear wheel is measured by a specially designed accelerometer. This instrument is mounted on the side of the truck body, although it obtains its motion through a rigid vertical rod extending down and connecting to the hub cap. The connection to the hub cap is made by means of a specially constructed, tight-fitting universal ball bearing. The deceleration is measured with this accelerometer by virtue of a small weight supported on a calibrated spring. A record is made on the moving paper tape by means of a pencil attached to the decelerometer weight. The vertical height of the record indicates the deceleration according to a scale, which depends upon the spring used under the accelerometer weight; and the cushioning qualities of the tire under test are indicated by the vertical height of the record. The horizontal scale of the record represents the position on the road. In this way the acceleration of the truck wheel may be accurately and definitely located on the record for any position on the road surface. On this same paper tape a record is also made by means of a special attachment between the rear axle and the body showing the deflection of the truck spring at any instant. Before testing, all truck springs are removed, cleaned, and greased, and are kept in this condition by being covered with a canvas boot. These springs are carefully calibrated for load and deflection under a testing machine. The total force delivered to the road surface is then the impact as determined from the deceleration and mass of the rear wheel (unsprung weight) plus the sprung weight as determined by the position of the truck spring at the same instant.

RELATION OF IMPACT TO STATIC LOADS

Each tire was subjected to a static-load test in a testing machine in which the load and deflection were recorded for several conditions of loading up to about 500 per cent overload and tire imprints or areas of contact were obtained. Although there is sometimes an indication of a relation between static and impact tests, a definite relation has not yet been established which can be applied to all tires for the purpose of predicting from the results of the static tests what the action will be under different conditions of impact.

The cushioning quality of a tire is measured by means of the accelerometer and has a direct relation to the deceleration resulting when the tire strikes the road surface with a given velocity. Undoubtedly the tire which offers the best cushioning qualities is one which will give the lowest deceleration value under the condition of impact. In other words, the tire which will cause the wheel to come to rest (in its vertical motion) in a greater distance will have a smaller deceleration value and consequently better cushioning qualities. These cushioning qualities of the different tires are dependent not only upon their inherent characteristics, but also to some extent on the character of the obstruction which they may strike. The horizontal speed with which the truck wheel strikes an obstruction or irregularity in the road surface and the load which it carries are also influencing factors. The cushioning quality of any particular tire is affected also to some extent by the tire temperature.

The cushioning quality of the tires as influenced by the tire temperature was carefully determined by mounting the tire on a truck wheel and placing it in a special impact machine wherein the tire was raised and dropped in a vertical position from different heights. The resulting deceleration was measured with an accelerometer. The temperatures were obtained by means of a water bath and electric heating units. The tire temperature was measured by means of thermocouples thrust into the body of the rubber composition, the temperature being varied from about 30° F. to 210° F. The results of these tests showed that the cushioning qualities were greater for hot tires than for cold tires and that the variation of cushioning with respect to the temperature was rectilinear.

CHARACTER OF ROAD SURFACES

The road runs of these truck-tire tests were always over the same selected stretches of different types of roads and, as far as possible, the truck wheels always followed the same line on the pavement. This resulted in a very good comparison. The two principal types of road surface used were a reasonably smooth concrete and a fairly rough granite (Belgian) block pavement. At the beginning of the tests a bituminous concrete road section was also used, but it was discovered in a short time that this changed in surface roughness to such an extent that successive comparisons could not be depended upon, and it was therefore eliminated.

The different types and sizes of trucks used in this series of tests included a 2-ton worm-drive Mack, a 3-ton Autocar, a 1-ton Ford with special gear reduction, a standard 5-ton class B Army truck, and a 6-wheel type 5-ton Army truck. These trucks all had additional wheel equipment whereby most of them could be equipped with solid, cushion, or pneumatic tires. The speed of operation varied from about 3 miles an hour to the maximum of which each particular truck was capable under the test conditions. None of the trucks exceeded a speed of 30 miles an hour.

ACTIVITY IN HIGHWAY RESEARCH

Figures which show the interest of highway builders in research as a means of solving highway problems are given in a list of highway research projects recently compiled by the Highway Research Board.

According to the list there are under way at the present time 479 active projects, 205 of which are being conducted by the various State highway departments and the United States Bureau of Public Roads, 184 by universities, and 90 by municipalities, counties, and industrial organizations.

These researches are suggesting more economical methods of design and are pointing the way to better methods of construction, according to A. N. Johnson, chairman of the Highway Research Board.

A DEFORMATION TEST FOR ASPHALTIC MIXTURES

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by H. M. MILBURN, Engineer of Tests

IN the design of asphaltic paving mixtures one of the first considerations is to produce a pavement which will retain its stability under high summer temperatures. Designers have been guided by past experience with the various classes of mixtures in producing a satisfactory pavement. There are no laboratory tests in general use for testing an actual specimen of the proposed mixture and it is felt that there is need for such a test.

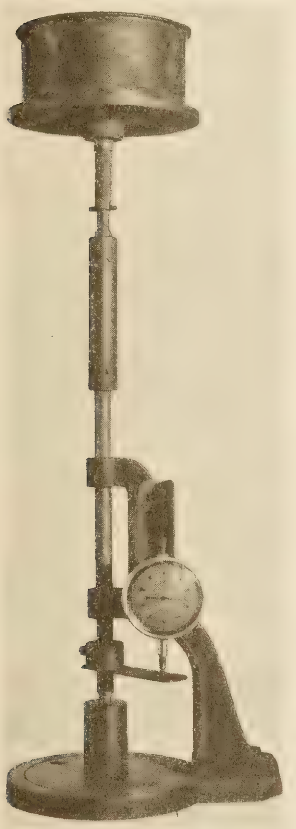


FIG. 1.—Apparatus used in making deformation tests

The Bureau of Public Roads has developed a test, making use of new apparatus which determines the effect of varying the percentage of asphalt in a mixture by subjecting compressed specimens to a constant load at a constant temperature for a definite time and measuring the deformation.

DESCRIPTION OF THE MACHINE

The deformation machine, shown in Figure 1, consists of the frame of a Vicat machine supporting a cylindrical rod in a vertical position. A cup for holding shot is attached to the upper end of the rod, the lower end being fitted into a cylindrical shoe 1 inch in height by 1¼ inches in diameter. Sufficient shot are placed in the cup so that the weight of the shot, cup, rod, and shoe is 5 kilograms. The decrease in height of the specimen is obtained by the use of an Ames dial reading to 0.001 of an inch attached to the frame, the foot of the Ames dial resting on a metal strip fastened to the rod of the machine. The size of the specimens tested is 1 inch in height by 1¼ inches in diameter. The machine is about 8 inches wide and 22 inches high. The deformation results given in this paper were obtained with this machine. Recently the machine has been redesigned in order to make it more compact, but in principle it is the same as the machine just described. This new machine, shown in Figure 2, differs from the former machine primarily in that two vertical parallel rods act as guides and the cup containing the shot is replaced by a rectangular weight. The machine is about 7½ inches wide and 12 inches high.

PREPARATION OF SPECIMENS

In the preparation of the specimens four different asphalts were used. Mixtures of each asphalt and a mineral aggregate consisting of 88 parts by weight of an asphalt sand and 12 parts by weight of a mineral filler were made. In three series of specimens the bitumen content was varied from 8 to 15 per cent by weight and in one series from 2 to 15 per cent, in each case the increments being 1 per cent. The mechanical analysis of the mineral aggregates used in the three series containing 8 to 15 per cent of bitumen and that used in the series containing 2 to 15 per cent of bitumen are given in columns (a) and (b), respectively, of the following table:

	(a)	(b)
	Per cent	*Per cent*
Passing a ¼-inch screen and retained on a No. 10 sieve	0.7	0.3
Passing a No. 10 sieve and retained on a No. 20 sieve	6.0	6.9
Passing a No. 20 sieve and retained on a No. 30 sieve	8.7	9.8
Passing a No. 30 sieve and retained on a No. 40 sieve	11.9	10.3
Passing a No. 40 sieve and retained on a No. 50 sieve	10.2	17.2
Passing a No. 50 sieve and retained on a No. 80 sieve	25.1	23.4
Passing a No. 80 sieve and retained on a No. 100 sieve	7.5	7.0
Passing a No. 100 sieve and retained on a No. 200 sieve	12.3	10.0
Passing a No. 200 sieve	17.5	15.1

In the preparation of the mixtures the asphalts and mineral aggregates were heated separately to a temperature of 300° to 325° F. and then thoroughly mixed in the proportions desired by hand, using a trowel or large spatula. In the making of the specimens a hollow steel mold 2½ inches in diameter and 3 inches in height was used. The diameter of the hole in the mold is 1¼ inches. The mold, which had been previously heated, was placed on a metal plate resting on the base of an Olsen testing machine and filled with an amount of the mixture sufficient to form a specimen substantially 1 inch in height. A cylindrical steel plunger 3½ inches long and $1\frac{15}{64}$ inches in diameter was then placed in the mold and pressure applied until the beam balanced at 2,000 pounds, using high speed. Then the balance weight was set at 10,000 pounds and the pressure applied at low speed until the beam just balanced. The pressure was then released and the specimen removed from the mold. The removal of the specimen was effected by placing the mold on a metal tube of sufficient height and diameter to allow free passage from the mold and forcing the specimen

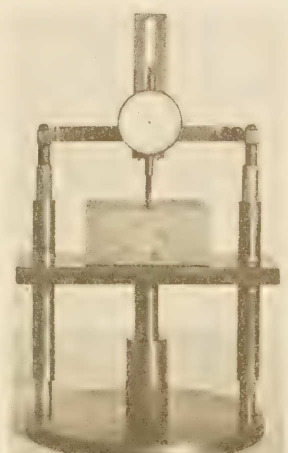


FIG. 2.—Improved design of apparatus to be used in future tests

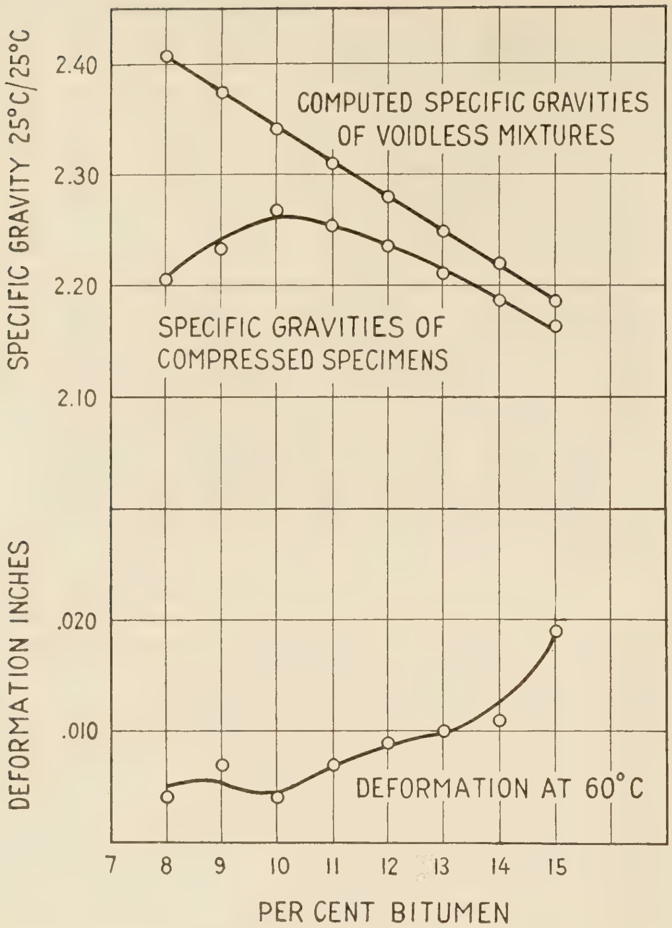


Fig. 3.—Results of series of tests on mixtures with asphalt L, having a penetration of 53

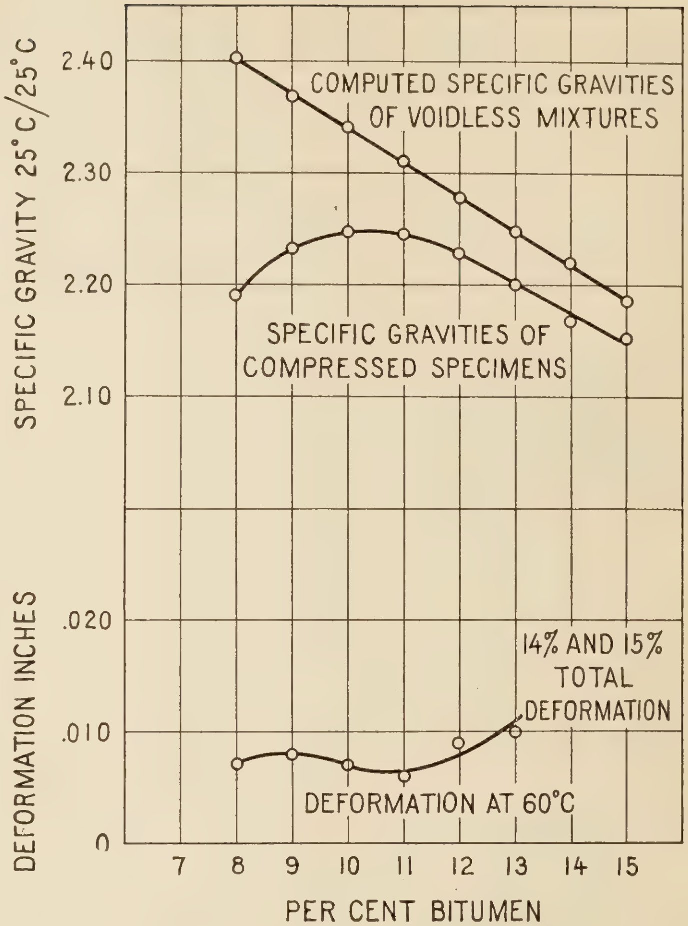


Fig. 4.—Results of series of tests on mixtures with asphalt J, having a penetration of 53

out by applying pressure to the plunger. The specific gravities of the specimens were determined by the displacement method.

While in these four series a pressure of 10,000 pounds was used exclusively, the pressure that was originally tried was 20,000 pounds as this pressure had previously been used with another mineral aggregate and specimens whose specific gravities seemed reasonable were obtained. (Results embodied in Progress Report of a Subcommittee of the Committee on Tests, A. A. S. H. O. for 1921.) In the present series, however, when the 20,000-pound pressure was tried, specific gravities which appeared abnormally high were obtained with specimens of low bitumen content. Furthermore, the specific gravities in general decreased from a maximum with 8 per cent of bitumen to a minimum with 15 per cent of bitumen, and analyses of the specimens with 8 per cent bitumen showed the mineral aggregate to contain 27.9 per cent of material passing the 200-mesh sieve, whereas the original mineral aggregate contained only 17.5 per cent. This increase in very fine material indicates that with this pressure of 20,000 pounds crushing of the mineral aggregate had occurred. It has been a matter of record[1] for many years that the voids in a sand decrease continuously to a certain point with the increase of mineral filler. It can be assumed, therefore, that the increase in fine material in the specimen had decreased the voids, resulting in high specific gravities.

[1] The Modern Asphalt Pavement, by Clifford Richardson, 2d edition, p. 82.

TESTING SPECIMENS FOR DEFORMATION

The deformation machine was placed in an oven and heat so applied that a temperature of 60° C. was registered by a thermometer embedded in a specimen on the base of the machine. The specimen to be tested was kept in the oven for one hour. The load was then applied and the deformation of the specimen was obtained by noting the original reading of the Ames dial and the reading at the end of two hours, the apparatus remaining in the oven at constant temperature. In general, two specimens from each mix were tested. The deformations of specimens which collapsed during the test are designated as total and no attempt was made to measure the deformation of such specimens.

DISCUSSION OF RESULTS

Figures 3, 4, 5, and 6 give the results of the four series of deformation tests with corresponding specific gravities of the specimens and the calculated specific gravity on the assumption of a voidless mixture. The difference between these two specific gravities indicates the amount of voids in the specimen. In Figure 3 the deformations decrease from that of the specimen containing the 15 per cent of bitumen to and including that of the specimen containing 10 per cent of bitumen; then an increase occurs for the specimen containing 9 per cent of bitumen with a subsequent decrease for the 8 per cent specimen. Figure 4 shows that the deformation decreases from the 15 and 14 per cent

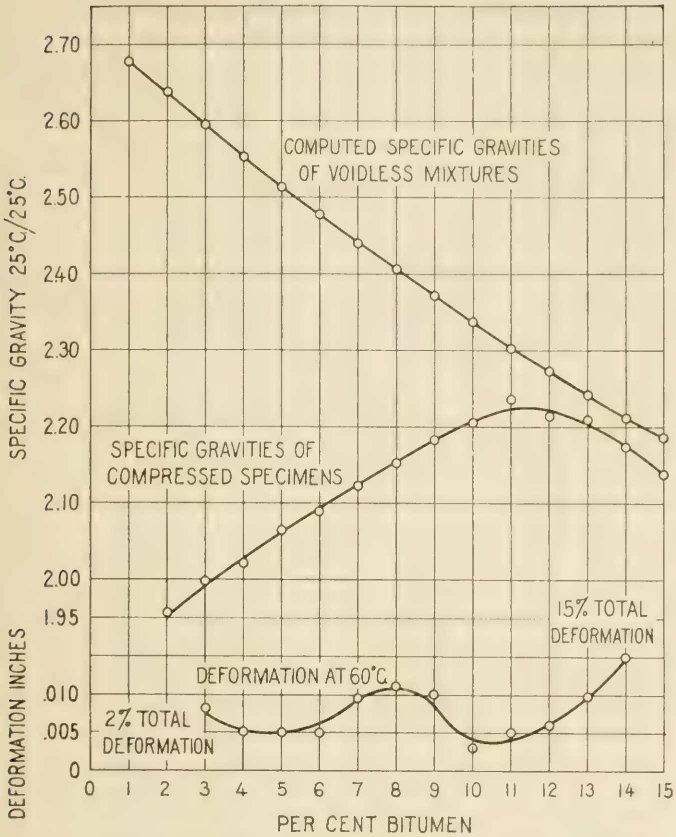


FIG. 5.—Results of series of tests on mixtures with asphalt I, having a penetration of 36

specimens to the 11 per cent one, inclusive, and then increases up to and including the 9 per cent specimen, with a subsequent decrease for the 8 per cent specimen. Figure 5 shows that the deformation decreases from the 15 per cent specimen to and including the 10 per cent specimen. An increase in deformation then occurs up to and including the 8 per cent specimen, a decrease then occurring with subsequent increase resulting in total deformation with the 2 per cent specimen. These results appear to indicate that there are two fields of comparatively low deformations, one field embracing specimens of high specific gravities and the other field embracing specimens of very low specific gravities. In a correlation of specific gravity determinations with deformation determinations for the purpose of ascertaining the proper amount of bitumen to use with a given mineral aggregate, the field of deformations embraced in the field of high specific gravities should be the mixtures to be given greatest consideration.

The deformations shown in Figure 6 decrease from a maximum for the 15 per cent specimen down to and including the specimen of lowest bitumen content. Consequently, to determine the best percentage of bitumen to use it would be necessary to correlate the results of the deformation tests with other physical tests, and it is believed that specific gravity results should be given considerable weight.

The specific gravities given in Figures 3, 4, 5, and 6 all show the same general characteristics, i. e., they all increase in general up to a maximum and then decrease. As no voidless specimens were obtained, the specific gravities in no instance are as great as the calculated specific gravities of the voidless mixtures. It is of interest to note that in no series does the maximum specific gravity represent a specimen containing the minimum percentage of voids, the specimen of lowest voids in each series being a specimen with lower specific gravity than the maximum.

CONCLUSIONS

Results seem to indicate:

(*a*) That the effect at high temperatures of varying the proportions of an asphalt and a mineral aggregate of the type used in the wearing course of a sheet asphalt pavement can be ascertained by subjecting compressed specimens of the mixture to a constant static load at a constant temperature for a definite time.

(*b*) That by a correlation of the specific gravity results of the compressed specimens with the deformation results data are obtained relative to the proper proportions to use of a given asphalt and a given mineral aggregate in the design of asphalt mixtures of the type used in the wearing course of a sheet asphalt pavement. It is realized, however, that a deformation test at a high temperature may not necessarily indicate the proper proportioning of the mixture in order that it may best withstand low temperatures. Under such conditions it seems necessary to develop physical tests for mixtures at a low temperature for correlation with specific gravity and deformation results.

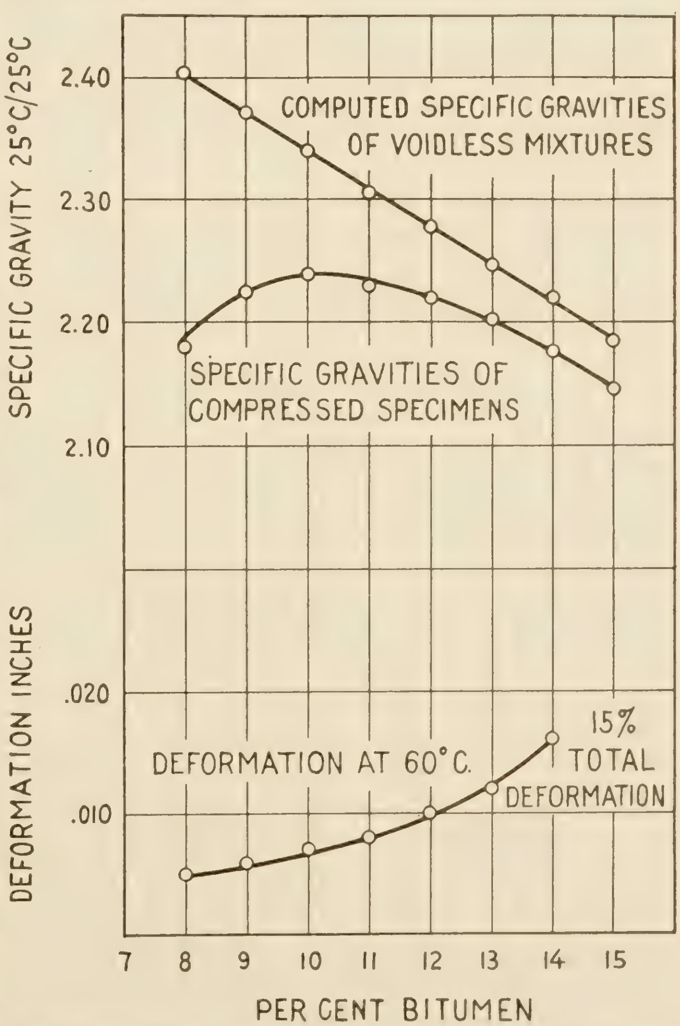


FIG. 6.—Results of series of tests on mixtures with asphalt K, having a penetration of 51

COLORS AND FORMS OF TRAFFIC SIGNALS

CODE PROPOSED BY SECTIONAL COMMITTEE OF THE AMERICAN ENGINEERING STANDARDS COMMITTEE

THE following is the text of a revised draft of the code of colors and forms for traffic signals for highways and vehicles submitted for approval to members of the sectional committee on code for traffic signals of the American Engineering Standards Committee.

The draft, which will probably be approved by the sectional committee, is as follows:

REVISED DRAFT OF CODE OF COLORS AND FORMS FOR TRAFFIC SIGNALS FOR HIGHWAYS AND VEHICLES

Section 1. Purpose and Scope

Rule 10. Purpose.

The purpose of this code is to provide a standard of colors and methods, as applied to highway traffic and vehicles, which will promote uniform usage and thus decrease the likelihood of accidents, and conserve human life and limb. The approval of specific signs, glasses, or pigments as complying with the specifications herein contained should be based upon tests made by competent and disinterested organizations having proper facilities and recognized standing, with final appeal to the United States Bureau of Standards.

Note.—To secure the uniform application of this code, enforcing officers are urged to consult the committee which formulated it (or the American Engineering Standards Committee, 29 West Thirty-ninth Street, New York City) regarding matters of interpretation or items of dispute.

Rule 11. Scope.

This code is intended to cover the use of luminous and nonluminous signs and signals in connection with highway traffic, including moving and flashing signals, the use of lights, semaphores, and other signaling devices on vehicles.

Section 2. Vehicle Lights and Signals

Rule 20. Headlights.

(*a*) Headlights shall be white, amber, yellow, or any intermediate hue. If electric headlights are used, they shall conform to the rules governing the approval of electric headlighting devices for motor vehicles, of the Illuminating Engineering Society (tentative standard of the A. E. S. C., D2).

(b) No red or green lights shall be displayed upon any vehicle so as to be visible from a point directly in front of it.

Rule 21. Tail lights.

Tail lights shall be red, as seen from the rear.

Note.—Red has been adopted on account of its present universal use for this purpose. This use is not considered consistent with the general standardization of colors, and if a change is made in any jurisdiction, yellow is recommended.

Rule 22. Warning signal lights.

(*a*) Light signals displayed on a vehicle to indicate the driver's intention to reduce speed or change direction should be yellow. They shall not be red or green.

(*b*) Nonluminous mechanical signals displayed to indicate the driver's intention to reduce speed or change direction shall consist of a semaphore which can be projected from the left side of the vehicle. It is recommended that the position of the semaphore when projected be limited to the horizontal.

Rule 23. Marker lights.

Marker lights to indicate the dimension limits of bodies or loads should be located as near the upper left front corner as possible and should be yellow and visible from the front and rear.

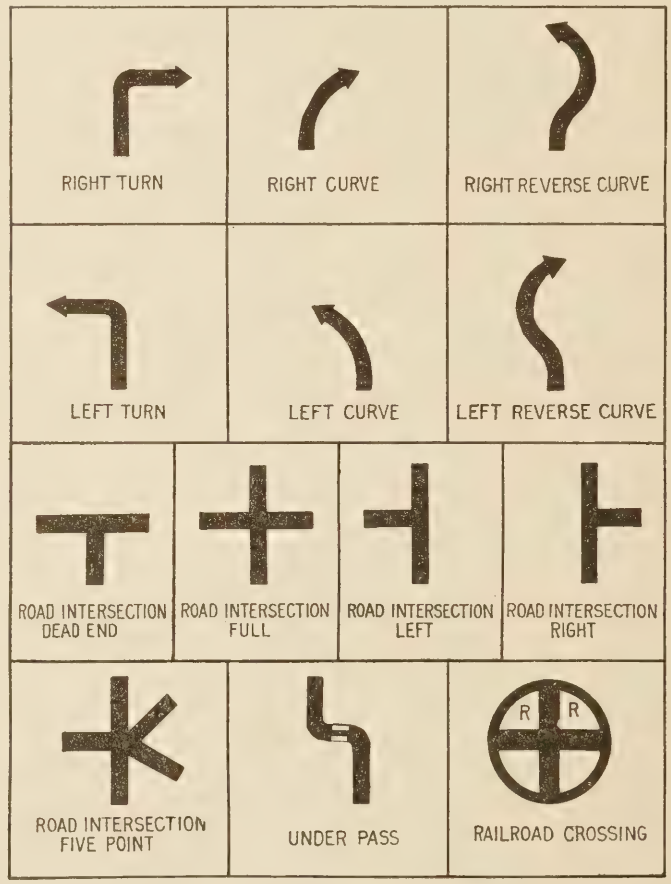


Fig. 1.—Symbols recommended by the sectional committee of the American Engineering Standards Committee

Section 3. Highway Traffic Signals—General

Rule 30. Significance of colors.

(*a*) Red shall be used as an indication to stop and for other purposes where required by law.

Note.—The use of red is proper as an indication to stop and to then proceed if conditions are favorable, as, for example, when "stop" and "proceed" regulations are in effect

(*b*) Yellow shall be used as an indication to exercise caution.

Note.—Yellow is appropriate when caution is to be exercised without stopping, as for partial street obstruction, so as to reserve red for a stop signal.

(*c*) Green shall be used as an indication to proceed.

Rule 31. Significance of form.

Where definite forms of luminous or nonluminous signs or signals are used either alone or in combination with colors to give the indications specified in Rule 30, the following will apply:

(*a*) Stop shall be indicated by having the greatest dimension horizontal, as with a horizontal semaphore or string of lights.

NOTE.—Where the word STOP is used to direct traffic, it should be in red letters arranged horizontally.

(*b*) Caution shall be indicated by having the greatest dimension at an angle of 45° with the horizontal when an inclined semaphore or string of lights is used.

NOTE.—Cautionary signs complying with Rule 33 may be used.

(*c*) Proceed shall be indicated by having the greatest dimension vertical, as with a vertical semaphore or string of lights.

NOTE.—Where the word GO is used to direct traffic, it should be in green letters arranged vertically.

Rule 32. Significance of flashing.

Flashing luminous signals shall conform to the use of colors prescribed in Rule 30.

Rule 33. Cautionary signs for drivers.

(*a*) Highway signs and signals of a cautionary nature for the notice of drivers shall have letters and symbols black on a yellow background or yellow upon a black background.

NOTE.—It is recommended that all cautionary signs have a background of one distinctive shape, and that such shape be reserved for this purpose.

(*b*) For signs to be read from vehicles in motion symbols are recommended in place of, or supplementary to, words.

NOTE.—When both symbols and words are used, symbols should be given greater prominence.

(*c*) When symbols are used, those illustrated in Figure 1 are recommended for the situation indicated. Other or special symbols should be such as not to be confused with these.

NOTE.—The symbol for the five-point road intersection is to be varied to fit each specific case.

Rule 34. Location of signs.

(*a*) Traffic signs shall be erected with the top not more than 8 feet above the surface of the roadway.

(*b*) Traffic signs shall be erected at the outer edge of the shoulder area and on right-hand side of roadway where practicable.

Rule 35. Cautionary signs for pedestrians.

Signs addressed to pedestrians shall consist of blue letters upon a white background.

SECTION 4. SIGNALS AT RAILROAD GRADE CROSSINGS

Rule 40. Aspect.

An electrically or mechanically operated signal used for the protection of highway traffic at railroad crossings shall present toward the highway when indicating the approach of a train the appearance of a horizontally swinging red light and (or) disk.

NOTE.—This covers the use of so-called wigwags and of alternately flashing red lights, and the use of these devices should be restricted to the purpose of indicating the approach of a train.

Rule 41. Location.

The railroad standard highway crossing sign and the signal shall be mounted on the same post.

Rule 42. Operating time.

Automatic signal devices for indicating the approach of trains shall be so arranged as to indicate for not less than 20 seconds before the arrival at the crossing of the fastest train operated on the track. Local conditions such as three or more tracks, bad approaches, etc., should be allowed for by increasing the operating time, bearing in mind that too long an operation by slow trains is undesirable.

Rule 43. Flashing light type.

(*a*) *Height.*—The lamps should preferably be not less than 6 feet nor more than 9 feet above the surface of the highway.

(*b*) *Width.*—The two lamps shall be mounted horizontally 2 feet 6 inches centers.

(*c*) *Flashes.*—Lights shall flash alternately. The number of flashes of each light per minute shall be 30 minimum, 45 maximum.

(*d*) *Hoods.* Lamp units shall be properly hooded.

(*e*) *Range.*—When lamps are operated at normal voltage, the range, on tangent, shall be at least 300 feet on a clear day, with a bright sun at or near the zenith.

(*f*) *Spread.*—The beam spread shall be not less than 3° each side of the axial beam under normal conditions. This beam spread is interpreted to refer to the point at the angle mentioned where the intensity of the beam is 50 per cent of the axial beam under normal conditions.

(*g*) *Lenses or roundels.*—The size shall be 5⅜ inches minimum, 8⅜ inches maximum.

Rule 44. Wigwag type.

(*a*) Length of stroke is the length of chord which subtends the arc, determined by the center of the disk in its extreme positions, and shall be 2 feet 6 inches.

(*b*) *Disk.*—The disk shall be 20 inches in diameter. Its field shall be white with a black circumferential border 1 inch wide. The horizontal and vertical diameters shall be shown by black lines 2½ inches wide. A red lens or roundel should be placed at the center in front of the lamp.

(*c*) *Number of cycles.*—Movement from one extreme to the other and back constitutes a cycle. The number of cycles per minute shall be 30 minimum and 45 maximum.

(*d*) The lamp, with which the disk shall be equipped shall be lighted when the disk is swinging.

Rule 45. Approach signal.

Advance warning signals which indicate approach to a railroad crossing, and not the actual approach of a train, shall conform to the provisions of Section 3, Rule 30.

SECTION 5. SPECIFICATIONS FOR COLORS

Rule 50. Definition of Colors.

Red, yellow, green, or blue as used in this code are intended to mean:

(*a*) The colors resulting from the transmission of the proper light through the proper glasses and having the characteristics described in Rules 51 and 52.

(*b*) The colors resulting from the reflection of white light from the proper pigments and having the characteristics described in Rule 53.

NOTE.—*For a definition of white light, reference is made to the report of the Colorimetry Committee of the Optical Society of America, 1920–21 (Journal of the Optical Society of America and Review Scientific Inst. 6, p. 563; 1922). Substantially, it is average sunlight at noon at latitude of Washington.*

Rule 51. Qualitative definition of colors for luminous signals.

(*a*) *Red.*—The spectrum of red shall contain both red and orange but not more than a trace of yellow and no green, blue, or violet. The most desirable hue is entirely free from yellow which means that the glass does not transmit the yellow light from a sodium flame.

(*b*) *Yellow.*—The spectrum of yellow shall contain red, yellow, and green, with but little blue and no violet. The most desirable hue is entirely free from blue and might be designated a light amber.

(*c*) *Green.*—The spectrum of green shall contain yellow, green, blue, and violet, with only a trace of red and orange. This hue is known as "admiralty green" and has a bluish tint when observed by daylight.

Rule 52. Quantitative definition of colors for luminous signals.

The colors red, yellow, and green shall have the following characteristics:

	Dominant wave length	Purity	Integral transmission of glass
	Millimicrons	*Per cent*	*Per cent*
Red	Not less than 624	Not less than 100	Not less than 10.
Yellow	Not less than 592 nor more than 600.	Not less than 97	Not less than 24.
Green	Not less than 496 nor more than 536.	Not less than 45	Not less than 11.

These values are determined by the transmission of light from a source at the color temperature of 2.360° K (practically that of the acetylene flame or present type of vacuum tungsten lamp at normal voltage) through the respective glasses. They are based upon spectral transmission measurements and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America.

NOTE.—The light and dark limits of the glasses on which the above values are based have the following relative transmissions on the scale of the American Railway Association.

	Light limit	Dark limit
Red	300	150
Yellow	200	100
Green	250	100

Rule 53. Quantitative definition of colors for nonluminous signs.

The colors red, yellow, green, and blue shall have the following characteristics:

	Dominant wave length	Purity	Integral reflection of pigment
	Millimicrons	*Per cent*	*Per cent*
Red	Not less than 608	Not less than 60	Not less than 8.
Yellow	Not less than 580 nor more than 588.	Not less than 80	Not less than 35.
Green	Not less than 524 nor more than 552.	Not less than 30	Not less than 8.
Blue	Not less than 466 nor more than 474.	Not less than 10	Not less than 4.

These values are determined by the reflection of white light from the respective pigments. They are based upon spectral reflection measurements under conditions of diffuse illumination and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America.

(Continued from page 123)

closer together than in the case of either the slump or flow test, indicating that determinations of consistency by this method are more independent of the type of aggregate than in the case of either of the other methods. In view of the fact that the compressive strength of the concrete appears also, in general, to be independent of the type of aggregate, this would appear to be an advantage. It will be noted also that the slope of the average curve for consistency as determined by the plate test appears to parallel in general the curve showing the relation between water-cement ratio and strength, It will be seen, for instance, that for the leaner mixes, such as 1:2½:5 and 1:3:6, the curves are much flatter than in the case of the richer mixes. The 1:2:3 mix shows the steepest slope and likewise the greatest range in consistency which is to be expected in view of the high plasticity or workability of a mix of this nature.

Observing the results of the flow and slump tests, we find that the points representing the various types of aggregates are much more widely separated. In the slump test, for instance, it will be observed that for a given water-cement ratio the points representing the tests with the gravel aggregate lie in general considerably higher than in the case of either the stone or slag concrete, while the stone in general is next and the slag lowest. These results are in accord with general experience with this test as a measure of consistency. The slump test appears likewise to be much more erratic for leaner mixes than for the richer mixes. This is also borne out by experience. The flow test results are fairly concordant for each type of aggregate. They, however, show the same general separation by groups as is indicated in the slump test.

For the purpose of checking the value of this device as a method of field control, a series of tests was run in connection with a concrete paving project. On this project rather poorly graded crushed limestone was being used as coarse aggregate, with a consequent tendency toward segregation whenever too much water was used. By the use of this device it was possible to control the mix so that a considerably drier consistency was obtained with consequent freedom from segregation, besides undoubted increase in strength.

The authors feel that enough work has been done with the plate device to warrant its presentation as a possible method of field control of the consistency of concrete. Extreme accuracy is not claimed. In no case is it possible to check closer than one or two pounds on the plate. However, the data which have been obtained indicate that this is sufficiently accurate for all practical purposes. The fact that the test gives practically the same results irrespective of the type of aggregate would appear to be a point in its favor, as would also the fact that, for a given mix, it appears to be a direct measure of the crushing strength of the concrete.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Report on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
*670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETIN

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JULY 31, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 32,459.80	$ 12,642.50	0.8	$ 15,417,487.54	$ 7,482,213.97	804.0	$ 300,107.34	$ 146,749.34	24.8	$ 2,290,029.33	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	237,926.80	145,397.06	7.1	2,049,599.63	1,270,492.41	157.3	176,977.10	108,150.70	26.7	2,011,711.89	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9				7,175,728.71	3,252,163.53	387.4	862,256.62	398,228.65	71.8	1,295,990.09	Arkansas
California	22,346,175.99	10,719,249.61	894.8				10,605,687.35	5,043,509.43	263.6	727,950.93	360,660.72	41.5	3,442,522.24	California
Colorado	11,876,703.94	6,067,814.34	651.2	47,462.26	26,635.82	4.0	4,072,242.93	2,043,234.77	184.8	999,617.76	503,000.34	54.8	2,292,432.73	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	305,300.83	96,200.00	4.8	1,795,870.72	550,037.75	28.1	371,356.48	137,522.73	5.5	1,252,866.86	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1				489,503.74	213,581.10	12.3	1,023,384.86	388,799.75	22.6	6,583.50	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				8,787,223.95	4,279,162.51	251.8	116,547.09	91,547.09	0.1	1,403,567.43	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	187,801.21	92,983.56	5.3	11,181,335.94	5,515,706.40	747.2	985,295.04	443,523.45	57.8	974,406.13	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	26,406.05	14,540.60	1.0	2,411,772.54	1,512,125.19	151.4	501,215.03	311,867.09	40.9	960,773.86	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2				8,350,804.09	4,041,970.89	281.2	487,835.78	243,917.89	17.4	3,702,005.94	Illinois
Indiana	13,639,172.85	6,562,455.68	422.1	227,828.34	104,962.96	7.6	15,692,088.38	7,615,237.24	455.5	953,429.82	467,680.58	26.5	1,500,748.54	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	102,280.50	39,495.95	16.4	7,425,789.49	3,275,441.22	418.3	978,898.49	477,595.00	99.7	2,506,507.84	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	769,544.49	299,522.80	17.6	11,987,431.14	5,098,719.21	582.7	996,527.64	388,438.30	54.5	1,831,695.37	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9				8,298,279.66	3,724,887.09	299.0	590,422.33	295,211.15	40.1	1,557,253.17	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	746,114.83	349,217.80	54.8	2,814,895.62	1,346,386.48	115.2	356,130.73	178,065.36	2.8	1,109,163.50	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	173,275.15	85,733.59	6.6	1,311,335.31	551,480.38	42.2				1,230,027.70	Maine
Maryland	8,132,506.90	3,849,383.15	294.4				2,070,460.66	884,007.47	65.7	1,139,721.78	546,339.26	75.2	5,003.12	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	198,726.73	53,820.00	2.7	5,378,944.39	1,411,819.74	82.5	716,821.54	226,335.00	13.4	1,850,261.98	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	894,484.74	419,344.47	50.9	14,227,346.89	6,656,371.05	432.4	694,848.44	276,168.19	0.6	3,424,798.38	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2				7,923,339.62	3,596,800.00	739.3	2,227,520.82	608,400.00	99.3	498,727.96	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	134,986.30	67,493.15	12.9	8,476,317.22	4,233,402.97	469.7	1,132,183.38	566,061.65	69.9	967,572.50	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,118.9	633,514.80	315,927.78	43.9	26,404,724.60	11,037,064.68	826.3	528,298.06	202,579.69	13.4	582,931.42	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	123,475.31	65,491.30	9.6	1,975,050.87	1,477,525.00	173.3	988,759.09	551,613.73	117.1	5,102,736.82	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	511,445.26	255,485.50	29.0	8,056,040.35	3,885,052.26	820.1	1,573,776.11	786,065.55	87.0	3,716,788.19	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	88,333.61	73,251.46	8.0	5,185,772.57	4,345,943.29	442.6	80,728.23	70,814.79	16.9	260,087.68	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1				1,177,495.61	556,510.59	37.7	30,791.75	12,495.00	0.8	245,356.54	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	183,976.37	48,630.00	3.2	9,289,268.96	3,311,777.62	67.3	1,345,330.48	304,408.53	21.2	38,832.86	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	491,965.02	413,672.15	29.5	3,834,381.19	2,441,722.52	365.1	383,518.39	239,309.08	29.1	1,765,723.64	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	2,380,670.38	934,192.29	59.7	28,262,125.75	9,454,545.10	578.0	9,832,600.00	2,693,351.45	178.6	5,054,078.63	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	483,841.87	173,882.00	18.1	7,658,702.09	3,323,201.00	184.5	2,245,098.85	978,672.58	68.5	771,208.83	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1,917.5	131,649.77	65,824.88	40.4	3,506,541.81	1,768,931.82	471.2	934,966.95	467,483.37	131.0	1,973,184.46	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	812,253.81	238,966.05	19.3	8,968,966.25	3,306,335.70	276.8	5,461,267.51	1,956,145.65	140.8	2,183,310.67	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	772,371.50	329,951.35	28.5	6,263,876.77	2,983,035.94	283.4	1,306,037.26	618,952.08	69.7	686,978.29	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	159,042.13	90,612.52	9.8	2,887,797.48	1,665,476.29	127.6	774,864.82	465,603.55	59.8	321,770.01	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3				32,354,835.50	8,595,365.14	560.3	7,213,528.55	2,152,295.42	162.4	992,054.47	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8				2,372,865.09	633,939.59	34.0	87,634.25	20,100.00	1.3	524,938.32	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	389,582.68	172,605.08	18.1	6,223,647.25	2,798,033.10	356.1	870,524.91	354,687.35	35.1	293,501.93	South Carolina
South Dakota	12,091,434 67	5,989,879.00	1,447.9	507,689.25	236,396.17	54.7	7,349,819.54	3,604,638.90	990.5	295,179.39	77,261.73	56.0	25,524.20	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	1,614,333.34	792,240.45	48.6	10,501,557.76	4,722,108.22	374.1	128,336.55	64,168.27	10.8	1,337,025.29	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	966,510.88	386,699.47	82.8	24,543,272.32	10,178,518.25	1,433.1	2,674,527.05	1,075,262.80	213.3	3,441,507.36	Texas
Utah	6,259,159.41	3,818,836.91	423.1	66,046.69	40,114.36	5.8	2,946,946.47	2,001,375.22	220.3	367,448.27	270,355.69	35.5	832,257.82	Utah
Vermont	3,015,174.51	1,452,894.45	107.8				1,219,635.45	584,290.42	26.6	488,824.92	164,815.73	10.3	.697,603.40	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	845,765.70	422,543.92	43.3	9,474,536.36	4,201,128.43	309.7	1,958,811.45	881,787.43	70.1	265,208.02	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	1,049,945.31	437,338.64	25.0	2,094,224.66	988,900.00	102.3	833,280.25	361,000.00	22.4	1,101,214.49	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	212,859.07	103,111.28	11.5	4,985,938.10	2,074,065.28	137.2	2,066,068.51	754,510.03	49.9	389,447.08	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1,451.7				4,514,450.66	2,211,353.25	213.8	1,208,300.43	596,037.00	74.3	3,824,728.13	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	198,443.81	126,165.00	18.8	3,724,227.32	2,334,689.53	254.2	480,564.59	309,065.00	38.0	113,281.80	Wyoming
Hawaii							342,277.22	97,440.00	6.5				633,910.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	16,708,314.59	7,531,091.91	800.1	384,062,463.52	168,181,717.94	16,644.2	60,498,115.62	23,593,103.74	2,559.2	73,289,740.41	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 99,453,213.05 Federal aid $ 45,298,750.40 Miles 4,367.9